PERFORMANCE OF A SCALABLE WASTEWATER TREATMENT PACKAGE UNIT AND POTENTIAL MILITARY APPLICATIONS

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CHAPTER 1
INTRODUCTION

1.1 CURRENT SITUATION

In the United States nearly 70 million (30%) housing units are served by septic tanks. The U. S. Environmental Protection Agency indicated in a survey conducted in 1978 that 32% of the population is still not being served by primary, secondary or advanced waste treatment facilities. It follows that several thousand homes in the State of Hawai`i are not provided with service connections to the county-operated wastewater collection, treatment, and disposal systems. Specifically, the island of Oahu has an estimated 128,659 cesspools and 1,027 septic tanks, Hawai`i maintains 29,084 cesspools and 1,697 septic tanks, Maui has 14,086 cesspools and 968 septic tanks, and Kauai has 4,197 cesspools and 868 septic tanks (Harold Yee, 1999). These homes (57% of Hawai`i's population) must each operate their own on-site system to collect, treat, and dispose sewage.

The U.S. Soil Conservation Service estimates that at least 68% of the land in the United States is unsuitable for leaching systems (Wenk, 1971 and Brewer 1978). Similarly, the EPA found that only about 32% of the United States has soil that will support leach field type systems (American City & County, April 1980). Unreliable soil absorption systems connected to septic tanks providing less than adequate treatment may cause contamination of the ground and

receiving waters. For the 1.1 million people living in the state of Hawai`i, there are a total of 176,026 cesspools and 4,560 septic tanks that are potentially delivering less than desired quality effluent into the ground or adjacent water bodies. Most of the existing systems are cesspools or septic tanks with attached leach fields, which provide only partial treatment at best. Hawai`i's Department of Health (DOH) is investigating alternative technologies to eventually replace cesspools and septic tanks where appropriate. Because cesspools pose a greater risk to groundwater contamination, they are a priority. Approximately 57% of the households in the state of Hawai`i are being serviced by an on-site cesspool or septic tank/absorption field system. Because of the relatively small and isolated land area of each island, this poses great concern. For example, an average family wastewater flow rate of 400 gallons per day, the septic systems and cesspools that have already been installed on the islands are discharging approximately 26 billion gallons of poorly treated wastewater each year. If not mitigated, Hawai`i's groundwater, ocean, and surrounding soil might be contaminated to dangerous and unhealthy levels.

The Hawai`i DOH is charged with protecting public health and has the authority to evaluate individual wastewater treatment systems. Because of health concerns, the DOH has a goal of eliminating cesspools by the year 2000 (Hawai`i Administration Rules (HAR) 11-62). The cost of infrastructure required to provide connections for all of the existing residences for the state of Hawai`i, which do not currently have connections to new or existing centralized

wastewater treatment facilities, is known to be extremely expensive (billions of dollars). Replacement of cesspools and septic tanks with individual wastewater treatment systems that provide complete treatment of wastewater to secondary or higher quality levels is an attractive alternative. A literature review shows that individual secondary-level treatment systems have been available on the U.S. mainland for some time and have a well established market and track record in Japan, Norway, and some European countries.

1.2 PURPOSE AND SCOPE OF WORK

To obtain permission from the Hawai`i DOH for installation of any individual aerobic treatment system, it must meet the National Sanitation Foundation (NSF) Standard 40 requirements (NSF, 1984) or be otherwise acceptable (Hawai`i Administrative Rules, HAR 11-62). The NSF 40 standard provides protocols for testing individual treatment units, as well as, criteria for acceptable minimum performance. Minimum performance for production of a NSF Standard 40 Class I effluent requires that the 30 consecutive-day mean effluent concentration of five day Biochemical Oxygen Demand (BOD_5) and Suspended Solids (SS) be no greater than 30 milligrams/liter (mg/l) and that there be at least 85% total removal of BOD. Additionally, the mean values of BOD_5 and SS for any 7-consecutive days cannot be greater than 45 mg/l. The effluent pH must always be between 6.0 and 9.0.

The main objectives of this project were:

- Conduct a literature review of studies similar in scope that have been conducted over the last 30 years.
- Evaluate the performance of a manufacturer supplied portable wastewater packaged treatment tank (BEST UCZ-5) using the NSF-40 testing protocol at standard operating conditions for future certification.
- Evaluate nutrient removal capabilities (nitrogen and phosphorus) and make recommendations for improvement, if needed.
- Discuss potential military applications of this unit in its daily operational commitments.

CHAPTER 2
THEORY

2.1 INTRODUCTION

Cesspools and septic tanks vary greatly from the process used by a Wastewater Treatment Package (WTP) unit to reduce sewage wastes. A WTP unit utilizes a combination of established unit processes that stabilize organic wastes and reduce or remove nutrients introduced into the tank. These unit processes include anaerobic treatment, aerobic treatment, settling, nitrification/de-nitrification, phosphorus removal, and disinfection. If maintained correctly, a WTP can provide excellent quality effluent on a continuous basis. Below is an introduction to these technologies and processes.

2.1.1 CESSPOOL

A cesspool is an excavation in the ground that is lined (usually with unmortared, open-jointed masonry), with a solid lid, and is covered with soil. A cesspool receives raw wastewater, retains solid waste materials within while permitting the liquids to seep out through the bottom and sides for treatment in the soil. Since cesspools are located well below the ground surface, they may interact with groundwater or be located in close proximity to aquifers used for drinking supply. Consequently, a potential hazard of cesspools is that any rise in groundwater level within a few feet from bottom of the cesspool can pose a health risk to any adjacent population. In any case, the effluent from cesspools

can travel to adjacent water supplies with little or no treatment in the soil. Because of this, many localities do not permit cesspool installation (Metcalf and Eddy, 1991 and Goldstein and Wenk, 1972).

2.1.2 SEPTIC TANK

Septic tanks have been around since the 1860's with little or no modification to the original design, except for the construction material used (Metcalf & Eddy, 1991). Early models were constructed out of redwood or steel, but these are no longer accepted by regulatory agencies. Today, they are constructed from thick-walled polyethylene, fiberglass material or concrete. A septic tank is a prefabricated tank that is an unheated-unmixed anaerobic digester that allows wastewater settleable solids to settle out, accumulating sludge at the bottom of the tank. Grease, oils, and material less dense than water float to the top of the tank forming a scum layer. The supernatant is released below the scum layer and above the sludge layer as effluent. After wastewater is processed through a septic tank, the effluent is typically distributed into a soil absorption field (leach field) where the effluent percolates through soil where most of the contaminants and nutrients are removed by soil bacteria. Table 1 lists typical septic tank/leach field performance.

Table 1: Typical Effluent Qualities Expected from Septic Tanks

Parameter	Raw waste (influent)*	Septic Tank (effluent)**	1.0 ft below bottom of leach field trench*	Sand Filter (effluent)**
BOD_5, mg/l of O_2	210–530	141-200	0	9
TSS, mg/l	237-600	50-90	0	6
Total Nitrogen, mg/l-N	35-85	25-60	N/A	25
Ammonia, mg/L-N	7-40	20-60	20	1
Nitrate, mg/L-N	<1	<1	40	19
Total Phosphorus, mg/L-P	10-27	10-30	10	8
Orthophosphate, mg/L-P	3-10	7-20	N/A	7
Fecal Coliforms, MPN/100 ml	10^6-10^{10}	10^3-10^6	0-10^3	0-10^2

* Metcalf and Eddy, 1991

** American City & County, 1980

Despite the large number of septic tanks in use, there have been only a few studies that accurately examine the extent of contamination or plume evolution in areas down gradient for such systems. A study conducted on a 44-year-old septic tank servicing a school in Ontario, Canada revealed that nitrate concentrations for the entire 110 meters (361 ft.) of plume mapped were above acceptable levels (Harman, Robertson, Cherry, and Zanini, 1996). At one location 100 meters from the tile bed, a nitrate concentration of 18 mg/L as Nitrogen was measured. Elevated nitrate concentrations likely extended beyond the school's property. A 75 meter long phosphate plume was also developed at the site although phosphate concentrations continued to be significantly attenuated in the unsaturated zone, from 9 mg/L at the source to 1.5 mg/L at the water table. Once in the water table, phosphorus levels seemed to be unattenuated for a distance of about 60 meters from the source before

concentrations decreased abruptly. The mobility of the phosphate plume at this site suggests that septic systems can be significant contributors of phosphorus and nitrate to nearby surface-water bodies. It was noted that this study was conducted under the worst case scenario. The site contained a majority of black water (toilet waste) with little dilution by wash water and a relatively fast ground-water flow velocity. These factors combined with long usage of the septic tank provided a near "worst case" condition for evaluation of solute transport in septic system plumes.

In the region of Langston, Ontario, septic systems probably contribute to the large number (30%) of domestic wells that are contaminated by nitrate.

2.1.2.1 SOIL ABSORPTION FIELD

Effluent from septic tanks is predominantly distributed via a piping network to a soil absorption disposal system. Soil absorption systems include conventional disposal fields, intermittent sand filters, disposal beds or pits, sand mounds, tile beds, and other types. The most common is the conventional absorption disposal field or so called leach field. A leach field percolates the effluent and aerobic and anaerobic soil bacteria treat the effluent. The network typically consists of a series of narrow, relatively shallow (2 ft in width by 5 ft deep) trenches filled with porous medium (usually gravel). The pipe should be laid with a slope of 1 to 2 inches for a 50 foot line, perpendicular to the groundwater flow. Effluent from the septic tank is applied to the disposal field by

intermittent gravity flow or by periodic dosing using a pump or a dosing siphon. The effluent enters the surrounding soil through placed perforated pipes where it enters the surrounding soil, into the vadose zone and eventually flows into the groundwater or to nearby waters sources. Treatment in the porous medium of the disposal field occurs through a combination of physical, biological, and chemical mechanisms. The porous medium acts as a submerged anaerobic filter under continuous inundation, and as an aerobic trickling filter under periodic application. A well constructed soil absorption field with the proper loading can provide excellent nutrient (nitrogen and phosphorus) reductions within several feet of soil depth.

When a pump or dosing siphon is not used, then intermittent or unsteady flow under gravity from the septic tank enters the disposal field. Under these anaerobic conditions, a biomat builds on the infiltrative surfaces of the disposal field. As the microorganisms metabolize the organic material from the septic tank, the thickness of the biomat will increase. After a long period of use, the disposal field will develop a dynamic equilibrium where effluent solids accumulate, biomass increase, mineralized constituents and particulate material biodegrade, which are carried away by the percolating liquid. The biomat that develops also acts as a mechanical and biological filter that often controls application rate, instead of the soil's permeability or capacity characteristics. The long term hydraulic capacity of the biomat is often termed the long-term

acceptance rate (LTAR), which have reported rates of 0.3 to 0.5 gal/ft^2/day depending on hydraulic head (Metcalf and Eddy, 1991).

The environment using a pump or dosing siphon is usually aerobic. As with other biological treatment processes, the biological treatment of organic materials occurs more rapidly under aerobic conditions than under anaerobic conditions. Because effluent is under pressure as it is being delivered to the disposal field, the effluent disperses over a larger area and the biomat formed is not as thick or uniform as in the intermittent gravity flow application. The thin and discontinuous biomass layer provides open areas in the soil that unsaturated conditions for effective treatment in the vadose zone can be attained.

No matter which type of delivery method is used, proper care in site assessment, design, and construction must be followed for effective and long term treatment. Properly designed and constructed disposal fields have an excellent ability to reduce fecal coliform, viruses, phosphorus, and ammonia concentrations within the first 3 feet of treatment depth (Metcalf and Eddy, 1991). Preliminary site evaluation, identification of site soil characteristics, percolation testing, hydrogeological characterization, analysis of assimilative capacity are all important factors for a maintenance free, well working disposal field.

2.2 WASTEWATER TREATMENT PACKAGE (WTP) UNIT

A WTP unit typically consists of a single tank that contains several individual chambers that will treat wastewater at varying loading rates and

hydraulic retention times (HRT). In sequence, WTP units begin with a settling chamber to perform discrete settling (Type I) and flocculant (Type II) settling of relatively heavy solids before entering the next chamber. The next chamber usually is a chamber to treat wastewater aerobically. This process is a biological assimilation process that consumes organic pollutants represented by Biological Oxygen Demand (BOD). Next, the treated wastewater is decanted into another chamber to provide final sedimentation before the effluent is discharged into receiving waters, a soil absorption field, or into a collection tank.

The UCZ-5, commonly referred to as the "The Tank" under this research project, modifies the above process by the addition of two anaerobic chambers before the aerobic chamber. The anaerobic chambers are used to increase overall BOD reduction and, because of the low synthesis rate of anaerobic microorganisms, the amount of sludge generated is minimal. A recycle line constructed into the tank directs aerobically treated water to the second anaerobic chamber to promote nutrient removal through de-nitrification. Within each anaerobic and aerobic chamber is an array of filter media designed to increase the surface area for attached growth of anoxic and aerobic microorganisms. The combination of suspended and attached growth processes increases treatment efficiencies of the tank. A small gap exists at the bottom of the tank between the settling and aerobic chamber, allowing settled sludge to gravitationally enter the aerobic chamber from beneath.

The UCZ-5 can be scaled up or down depending on how many people will be served by the tank. BEST Incorporated used the number 5 to designate the number of People Equivalence (PE) the UCZ-5 was meant to serve. Tank dimensions and mechanical devices (blower) can be adjusted to accommodate typical numbers of PE anticipated.

2.2.1 ANAEROBIC TREATMENT

Anaerobic treatment is a biological unit process that utilizes a culture of anaerobic microorganisms to degrade organic solids into gas and new biomass. The biomass created using this process is a stabilized sludge with a reduced amount of organics and pathogens. The sludge can be disposed into a sanitary landfill or can be used for composting or fertilizing after the solids have been dewatered and dried. The sludge in the tank is required to be removed during regular maintenance periods, about every 6 months. Anaerobic treatment is typically a three-step process. First, microorganisms hydrolyze higher molecular mass compounds, such as solids (food particles, paper, feces, etc.), polymers and lipids, from the influent into smaller compounds such as amino acids and monosaccharides which can be used directly as an energy source for other bacteria. Next, acidogenic, or acid forming organisms create a waste product of volatile fatty acids, hydrogen, and carbon monoxide utilizing a fermentation process. The most common of the acids produced is acetic acid. But, propionic, butyric, and valeric acids are also produced or are present (Eckenfelder & Ford,

1970). In the third step, methanogens, methane forming organisms, are used to convert acetic acid, hydrogen and carbon monoxide into methane and carbon dioxide gas. Not all anaerobic digesters contain methanogens, in which case, they produce primarily CO_2. This is the case for the BEST Tank. The product remaining is a stabilized organic sludge. All of the organisms involved in anaerobic digestion live in symbiosis where the survival of one organism depends on the other. The methanogens depend on the acidogens to produce an energy source, while the acidogens depend on the methanogens to remove hydrogen which inhibits the acidogens' cell growth (Metcalf & Eddy, 1991). The methanogens are sensitive to pH. Therefore, an adequate alkalinity must be maintained within a range of 6.7 to 7.4.

Some disadvantages of anaerobic digestion include a long retention time (ranging from 10 to 60 days), and tight control of temperature and pH. However, the slow bacterial growth rates help to stabilize the sludge completely. Most of the pathogens are destroyed and at least half of the organic content is degraded with a small amount of produced biomass.

The first two chambers incorporated inside The Tank are anaerobic chambers that, in theory, will stabilize the organic matter contained in the wastewater before the wastewater continues on to the third chamber to be treated aerobically. These two chambers contain filter media that help provide a long cell-retention time (CRT) in spite of a relatively short hydraulic retention times (HRT). Because the bacteria are retained on the media inside the two

anaerobic chambers, mean CRTs on the order of 100 days can be obtained. Large values of CRTs can be achieved with short hydraulic retention times, so the anaerobic filter can be used to treat low-strength wastes at ambient temperatures. In the anaerobic filter process, the water is typically pumped upwards through a column, contacting the medial on which anaerobic bacteria grow and are retained on which a biological growth has been developed. Removal efficiencies of 75 - 90% can be achieved with hydraulic detention times between 2 and 10 hours (Metcalf and Eddy, 1991). The Tank uses the same principle, except instead of the wastewater being pumped upward through the filter media, the wastewater is forced downward by the pressure differential caused by incoming influent and the water level between chambers.

2.2.2 AEROBIC TREATMENT

The Activated Sludge (AS) process is the most common aerobic suspended-growth biological treatment process used to remove organic materials from wastewater. The AS process was developed in England in 1914 by Ardern and Lockett and was so named because it involved the production of an activated mass of microorganisms capable of stabilizing a waste aerobically (Metcalf & Eddy, 1991). Today, many variations from the original process exist: Continuous Flow Stirred-Tank Reactor (CFSTR), Plug Flow Reactor (PFR), and Sequencing Batch Reactor (SBR) designs. These processes can be operated with or without recycle and with or without "wasting the sludge" to control CRT

and to control effluent quality. Operationally, the wastewater enters a tank where an aerobic bacterial culture is maintained in suspension. This mixture is called a "mixed liquor" because the wastewater containing organic matter is mixed with the existing suspended culture. Suspended culture of aerobic microorganisms then convert the organic material present in the wastewater into new biomass, carbon dioxide gas, and water. There are two steps in the activated sludge process (1) bio-transformation and (2) solids removal or clarification. Bio-transformation occurs as wastewater enters the aeration tank where suspended and dissolved organic material are sorbed onto cells and into flocs and then metabolized. The suspended culture is held in suspension usually by diffused or mechanical aeration. The mixed liquor passes into a clarifier where separation occurs. The supernatant effluent can then be further treated using a tertiary treatment process, if required. The settled cells are recycled back to the entrance of the AS chamber to maintain the desired concentration of organisms. In most processes a fraction of the recycled cells is wasted to maintain desired kinetics for treatment. The conversion of organic material (for any aerobic process) in the wastewater is as follows (Metcalf & Eddy, 1991):

Oxidation and synthesis:

$$COHNS + O_2 + \text{nutrients} \xrightarrow{\text{Bacteria}} CO_2 + NH_3 + C_5H_7NO_2 + \text{other end products } (H_20, PO_4^{3-}, NO_3^-)$$

Endogenous respiration:

$$C_5H_7NO_2 + 5O_2 \xrightarrow{\text{Bacteria}} 5CO_2 + 2H_20 + NH_3 + \text{energy}$$

The COHNS represents the organic material contained in the wastewater and the $C_5H_7NO_2$ represents the new cells produced from bacterial conversion. Organic material come from human wastes (excretions and urea) and washing wastes from cooking, food processing, soaps, etc. Although this process is not designed to remove inorganic or suspended materials, AS will also remove them effectively. During the respiration phase, new cells are converted to simple and stable end products. Approximately 30 - 50% of organic material are converted into new biomass, while the balance is converted to carbon dioxide and other end products.

The AS process can also aid in the removal of unwanted nutrients in the wastewater. Nitrogen can be removed from the wastewater by specialized autotrophic bacteria, which convert ammonia-nitrogen to nitrite and then to nitrate (nitrification). The wastewater can then be treated anaerobically to convert nitrate to gaseous nitrogen (denitrification), thereby reducing the overall nitrogen content in the wastewater. Phosphorus can also be removed using alternating aerobic/anaerobic staged reactors. Microbes utilizing phosphorus during cell synthesis and energy transport can result in 10 to 30% phosphorus removal during the secondary biological treatment (Metcalf & Eddy, 1991).

In attached growth treatment process, a biological slime layer grows on an

inert media (plastic in the case of the UCZ-5). Organic matter in the wastewater is adsorbed into the slime layer and degraded by aerobic microorganisms in the outer portion of the slime layer. As the microorganisms grow, oxygen is consumed before reaching the full depth of the slime layer, producing an anaerobic environment near the media face. As the depth of the slime layer increases, the organic matter is metabolized before reaching the bacteria near the media face. These bacteria subsequently lose their ability to cling to the media surface and are sheared from the media by the wastewater passing over the media. Attached growth processes are more difficult to model than suspended growth processes because of the unpredictable growth and hydraulic characteristics (Foree, 1981). The Tank uses an aerobic submerged filter to promote attached growth treatment. Air is introduced using a blower via an air hose and plastic pipe diffuser positioned below the plastic filter media.

2.2.3 SEDIMENTATION

Sedimentation is a physical process that separates suspended particles from the water by gravitational force. Sedimentation or settling is one of the most widely unit processes used in wastewater treatment. Sedimentation is used for grit removal, particulate-matter removal in the primary settling basins, biological-floc removal in activated sludge settling, and chemical-floc removal when chemical addition is used for treatment (Metcalf and Eddy, 1991). Settling is characterized on the basis of the solids concentration and their tendency to

interact: (1) discrete, (2) flocculant, (3) hindered (also called zone settling), and (4) compression settling.

The Tank is designed with a small settling chamber that is used for the collection of settleable solids sloughed from the aerobic filter media (chamber #3). Solids accumulation in the settling chamber can be caused from sloughing during normal operations or from back washing the filter media during periodic maintenance periods. It is anticipated that flocculant, hindered, and compression settling are taking place within the chamber. A settling analysis was not part of this study.

2.2.4 NITROGEN REMOVAL

Nitrogen exists in many forms naturally, and these are constantly being converted from one form to another. Wastewater usually contains organic nitrogen, ammonia, nitrate and nitrite. They are interrelated in that each is a form of nitrogen, which is a vital nutrient for survival, but must be in certain forms for use by living organisms. The largest reservoir of nitrogen is in our air supply. Air is 78% nitrogen, but it is not usable to humans. This gaseous nitrogen is used by plants, which are in turn used by animals for food essential for survival. After death, the organic nitrogen is converted during decay and decomposition by microorganisms into ammonia. This process is called ammonification. Plants assimilate some of the ammonia, but the majority of it is converted into nitrite and then nitrate through nitrification by microorganisms. Some of the nitrate is

converted back into nitrite through denitrification by microorganisms under anoxic conditions and some is used by plants and animals, putting the nitrogen back into organic form. Nitrite and nitrate can also go back into the gaseous forms through denitrification by microorganisms.

Approximately 60% of the total nitrogen present in raw domestic wastewater in the U.S. is composed of ammonia-N (Tchobanoglous and Schroeder, 1985) and the remaining 40% is organic nitrogen. Ammonia-N can exist as ammonium ion (NH_4^+) and free ammonia (NH_3), which is the most reduced form of nitrogen in nature. Ammonia is naturally present in all surface waters and wastewater. Because of its oxidation state of -3, it has a large oxygen demand. Inputting large amounts of ammonia into a receiving stream could deplete its dissolved oxygen. The ammonia ion also acts as a biostimulant. An unpolluted lake or river is usually nitrogen poor. Algae growth is promoted with the input of ammonia, causing odor and taste problems with the water. NH_3 ammonia is also toxic to aquatic life above concentrations of 0.2 mg/L.

Nitrification is a biological process that converts ammonia into nitrite and then to nitrate, which is the most oxidized form of nitrogen. This is accomplished by special autotrophic bacteria called Nitrosomonas and Nitrobactor under aerobic conditions. Autotrophs are bacteria that use carbon dioxide (CO_2) for a food source and ammonium ion (NH_4^+) for energy. Nitrosomonas oxidizes ammonia to the intermediate product nitrite and nitrite is converted to nitrate by

Nitrobactor. Nitrification will not occur unless proper conditions for bacterial growth are established. NO_3^- in drinking water is a potential health hazard that can cause methemoglobinemia (blue baby syndrome). Because of the potential health risk, the EPA has set regulations on NO_3^-, as well as NO_2^-, at or below 10 mg/L as N and 1.0 mg/L as N, respectively. NO_3^- and NO_2^-, are also biostimulants, which promote algae growth in lakes and streams, depleting dissolved oxygen.

Both nitrate and nitrite are almost always undetectable in raw and primary wastewater because it is anaerobic and the nitrifiers require high concentrations of oxygen. All of the nitrogen in raw and primary wastewater is in the form of ammonia and organic nitrogen. For secondary effluent that has been nitrified most of the ammonia has been converted to nitrate, with some intermediate nitrite remaining. Typical nitrate and nitrite concentrations for secondary nitrified effluents are 15-30 mg/L as N and less than 0.1 mg/L as N, respectively.

Removal of nitrogen (denitrification) from water and wastewater is usually a tertiary treatment process. Bacteria convert NO_3^- and NO_2^- to nitrogen gas (NO, N_2O, and N_2) when molecular O_2 is not present (anoxic conditions), reducing the amount of total nitrogen in the wastewater. Maintaining the pH between 7 and 8 and keeping anoxic conditions are important to healthy bacterial cultures responsible for denitrification. The anaerobic bacteria obtain energy for growth from the conversion of nitrate to nitrogen gas but require a

source of carbon for cell synthesis. The Tank uses incoming raw wastewater for its carbon source while the recycle line provides nitrate for denitrification.

2.2.5 PHOSPHORUS REMOVAL

Phosphorus is an important consideration in the treatment of wastewater because it is a biostimulant resulting in a depletion of oxygen in receiving water. Phosphorus can only be synthesized by humans, plants, and microorganisms as orthophosphate (PO_4^{3-}). All other forms of phosphorus (condensed phosphates and organically bound phosphorus) must be converted through hydrolysis to orthophosphate to become usable to living organisms. Phosphorus infiltrates our waterways in several different ways. Phosphorus typically enters wastewater from body wastes, food wastes, and household detergents (Foree and Nicholas, 1981). Polyphosphates are used in public water supplies to control corrosion, used in boiler operation to prevent scaling, used in softening water to stabilize calcium carbonate to eliminate the need for recarbonation, and used in agriculture as a nutrient for plants. With the benefits of phosphorus, some bi-products are produced that need to be treated before re-entering our environment.

Phosphorus discharge has demonstrated to be essential for the growth of algae and cyanobacterial algal blooms, which can deplete the dissolved oxygen in the water source causing taste and odor problems, if not corrected. The expected total phosphorus content in raw municipal wastewater varies from 4

mg/l to 15 mg/l (8-mg/l average) during dry weather periods. Total phosphorus is composed of organic and inorganic phosphorus. Typical values of organic and inorganic phosphorus are 1 to 5 mg/l and 3 to 10 mg/l, respectively. Secondary treated effluent ranges from 3 to 10 mg/l of total phosphorus.

Conventional treatment processes were inadequate to meet the standards and so chemical precipitation of phosphorus was common until the development of Biological Phosphorus (Bio-P) removal systems. Bio-P removal was developed when it was identified that enhanced phosphorus storage by bacteria was possible when they were exposed to alternating anaerobic and aerobic environments. Biological phosphorus removal systems can reduce effluent concentrations by 70 to 80 percent (Kiely, 1996).

The Bio-P process used by The Tank to reduce phosphorus is similar to the A/O process. In the A/O process, the settled sludge from the aerobic chamber is recycled back to the anaerobic chamber. Under anaerobic conditions, the phosphorus contained in the wastewater and the recycled cell mass are released as soluble phosphates. In The Tank, anaerobically treated wastewater enters the aerobic chamber upon the influence of head differential caused by influent. After aerobic treatment, the wastewater is recycled back to the anaerobic chamber. This cycle continues until the wastewater is replaced by the influent.

2.2.6 DISINFECTION

Disinfection refers to the selective destruction of disease-causing organisms where sterilization refers to the destruction of all organisms. Chlorine is the most common disinfectant used in the United States to neutralize organic matter in our drinking water and wastewater. Organic matter produces a chlorine demand that must be met or exceeded to ensure destruction of disease causing pathogenic microorganisms. Chlorine exists in many forms including free chlorine (Cl_2 + $HOCl$ + OCl^-), and combined chlorine or chloramines (NH_2Cl + $NHCl_2$ + NCl_3), mono (NH_2Cl), di ($NHCl_2$), tri (nitrogen trichloride (NCl_3)). Total residual chlorine includes the combination of free and combined chlorine. Typically, chlorine is added to drinking water beyond the required amount to form a residual concentration allowing complete disinfection by the time the treated water reaches its of point of delivery. If ammonia is present in the water (as in the case of wastewater), it will also consume chlorine. Therefore, additional chlorine will be required to compensate for the ammonia consuming chlorine.

Besides using the chlorine's oxidizing ability to inhibit enzyme activity to disinfect, other mechanisms also cause disinfection. These mechanisms include: (1) damage to the cell wall, (2) alteration of cell permeability, and (3) alteration of the colloidal nature of the protoplasm. Damage or destruction of cell wall will result in cell disintegration and death. Some agents, such as penicillin, inhibit the synthesis of the bacterial cell wall. Agents such as phenolic compounds and detergents alter the cell permeability by destroying the

cytoplasmic membrane. By destroying the membrane, nitrogen and phosphorus are allowed to escape causing death to the cell. Heat, radiation (UV), and highly acidic or alkaline agents alter colloidal nature of protoplasm. Heat will coagulate the cell protein and acids or bases will denature proteins, producing a lethal effect (Metcalf and Eddy, 1991). The UCZ-5 is constructed with a 22 liter (5.8 gal) disinfection chamber that provides approximately 20 minutes of chlorine detention time at 400 GPD. The wastewater flows into the disinfection chamber after it makes contact with chlorine tablets contained in a specially designed canister. A disinfection study on the UCZ-5 was not conducted under this study; however, research was conducted under a separate study (Edling, 1999).

CHAPTER 3
CASE STUDIES

3.1 BOYD'S COUNTY DEMONSTRATION PROJECT, USA

The cost of providing a municipal collection system connection for domestic wastewater is between $8,000 and $10,000 per residence (Waldorf, 1981), which does not include the monthly service charge for treatment. Because of these prohibitive costs, low population densities, and severe topographical problems experienced in the rural Appalachian area, not to mention the rest of the United States, a study was conducted by the Boyd County Sanitation District #3 and financed by the Appalachian Regional Commission (ARC). The project site was located in the northeastern part of Kentucky (Foree, 1981). The purpose of the study was to evaluate the ability of aerobic treatment systems and associated disposal techniques to provide effective wastewater treatment. The project consisted of installing 4 different types of package aerobic treatment systems at 22 different sites, providing treatment for a total of 26 homes. Out of the 22 sites, 10 units used surface discharge, 10 units used lateral field (leach field), 1 site used an Evapotranspiration (ET) Bed, and another site used a recycle with a lateral field to dispose their effluent. Mixed liquor and effluent samples were collected monthly from two Bi-A-Robi, three Cromaglass, four Eastern Environmental Controls Incorporated (EEC), and three Multi-Flo units during the study. Effluent samples were measured for Total Suspended Solids (TSS), Dissolved Oxygen

(DO), pH, BOD_5, ammonia, nitrate, and phosphorus concentrations. Influent characteristics were not measured but were assumed to be approximately 250 for BOD_5 and TSS and 50 mg/L for total nitrogen concentrations. Table 2 summarizes the results from Boyd's County demonstration study:

Table 2: Average effluent concentrations from Boyd County Demonstration Project.

Unit Type	TSS (mg/L)	DO (mg/L)	pH	BOD_5 (mg/L)	NH_3^+-N (mg/L)	NO_3^-N (mg/L)	Total P (mg/L)
Bi-A-Robi	283	5.9	7.7	86	71	5	15
Cromaglass CA-5	104	4.9	7.7	70	45	9	15
Cromaglass CA-900	61	3.4	7.2	27	1	50	15
EEC	88	4.2	7.4	44	20	20	14
Multi-Flo	31	4.2	6.4	6	2	65	17
Multi-Flo (multi-family	52	6.3	7.5	40	2	21	11

* Average data values are from August 1979 through July 1980.

Capital and operational costs were also examined. Capital costs for treatment units ranged from $1,618 for a Bi-A-Robi unit to $4,297 for the EEC 54291-7.5 unit. These costs (which are in 1980 dollars) include the aerobic unit, freight, installation and septic tank, if required. The Bi-A-Robi and the EEC Mini-Plant required a septic tank to be attached, preceding the aerobic treatment unit. Unit purchase, installation, repair, maintenance, travel, and power consumption costs were amortized over a 20 year period at a monthly interest rate of 1%, which is equivalent to an effective annual interest rate of 13%. These amortized costs ranged from $93.26 to $106.50 per month, resulting in an average cost of $95.98 with a standard deviation of $8.55 per month. These costs are in 1980

dollars. The adjusted cost for 1999 using an average annual inflation rate of 4 percent would be approximately $130.00 per month. Some general conclusions derived from this study were:

- Proper unit installation, attention to site climatic limitation in selecting disposal methods, and consistent preventative maintenance are crucial to achieving good effluent quality at reduced costs.
- Multi-family units show particular promise for reducing wastewater treatment costs for homes short distances apart.
- Some type of central management authority is necessary to effectively install, operate, and maintain aerobic treatment plants.
- Phosphorus was not removed by any of the units and chemical precipitation of phosphorus does not appear to be feasible.
- Soil absorption appears to be the best means of removing phosphorus for on-site systems.
- Sand filtration is highly recommended to assure removal of suspended solids.
- Disinfection, if necessary, should follow the sand filter with adequate contact time before surface discharge.
- Disposal of sludge and screenings is a problem, which must be solved.
- Kinetics for complete-mix treatment systems adequately predict treatment efficiencies.

3.2 NARITA CITY, JAPAN

The Sewerage Issue Liaison Committee, Tokyo University, in cooperation with Kurashi-no-Techo Incorporated and OM Research Institute conducted a study from November 1993 to July 1994 on 10 different makes of "Joint Wastewater Treatment Systems", including a UCZ-10 manufactured by BEST Industries (Hamada and Nakanishi, 1994). Supernatant from a primary sedimentation pond that followed an activated sludge process from a local Japanese wastewater treatment plant was supplied to each of these 10 tanks. The average flow rate was 740 GPD (2,800 L/day) with influent BOD_5 concentration between 200 and 250 mg/L and a total nitrogen concentration of approximately 40 mg/L-N. Effluent transparency was measured 3 times a week and pH, BOD_5, COD, Nitrogen, and Phosphorus was measured monthly over the 8 month period. Table 3 lists the average effluent results.

Table 3: Effluent Results of Full Scale Survey of Joint Treatment Systems

No	Manufacturer	Total Effective Capacity (gal)	BOD (mg/L)*	Ammonia (mg/L)	Total Nitrogen (mg/L)	Ratio of Days Transparency < than 30 cm (>%)**
1	Fuji Clean	1691	17	14	20	60
2	Best Industries	1744	15	16	21	20
3	Homer	1691	28	26	29	70
4	Nishihara Neo	1664	21	18	24	50
5	Komatsu	1691	18	16	20	50
6	Kubota	1664	21	24	25	60
7	Miyoshi	1664	23	18	22	30
8	Hitachi	1664	35	27	30	80
9	Taitech	1981	15	15	19	0
10	National	1691	23	28	29	60

*BOD figures only include data not affected by effluent contaminated by scum during testing.
**Desired transparency for Japan is greater than 30 cm.

In Japan, turbidity is measured by pouring water in a one meter long glass cylinder with a double cross marked at the bottom of the cylinder. Water is then slowly removed until the double cross marking is clearly seen by an observer looking down from the top of the cylinder. When the marking can be seen, the water level is recorded as the transparency in cm.

A related study pertaining to BOD and total nitrogen impacts as a function of varying internal recycle rates was conducted on same unit used in the present research (the UCZ-5 manufactured by BEST Industries), which also provides good baseline data (Xie, Kondo, and Okabayashi, unpublished). The average influent flow rate used was 60L/hr (380 GPD) with an inflow peak in the afternoon. The average effluent BOD was 28 mg/l without circulation and 21 mg/l with circulation - a 25% decrease in effluent BOD with circulation. Total nitrogen averaged 28 mg/l without circulation and 22 mg/l with circulation - a 21% improvement. The conclusion of this study was that circulation of the treated waste and sludge from the contact aeration chamber to the anaerobic chamber No. 1 provides positive effects on BOD removal efficiency and stabilization of the effluent quality, especially on nitrogen removal rate.

An analysis of power consumption was also given. Electrical power is required to operate a blower that provides aeration for the aerobic chamber. The energy requirements ranged from 100 Kilo-watt-hour (kWh) for the BEST system

to about 63 kWh for the National system. Even though the tank manufactured by BEST Industries was slightly more expensive to operate than the others, the tank manufactured by BEST surpassed all other tanks in providing the best overall effluent quality and was less expensive to purchase than most of the others.

To make this program attractive and feasible, Narita's City Counsel makes a special effort to install these packaged treatment systems via a grant for installation, maintenance, and cleaning. The grant provides $11,000 for installation and $330 per year for the maintenance and cleaning program. The total cost per installation is about $20,000 with yearly maintenance and cleaning costs of $160 and $580, respectively. The owner pays half the installation cost and the balance of the maintenance and cleaning cost, which is equivalent to a monthly sewer treatment charge of about $35 per month. An exchange rate of $100.00 per yen was used to convert yen into dollars. This innovative idea was effective and gave a clear message to the public of Narita City. All citizens paid an equal sewage fee regardless of where they lived and what type of system was installed, provided consistent effluent quality, and created an incentive for people to invest in a packaged treatment unit living in rural and agricultural areas.

3.3 SEQUENCING BATCH REACTOR (SBR) ACTIVATED SLUDGE PROCESS, JAPAN

The Sequencing Batch Reactor (SBR) process is an effective method to treat domestic wastewater. In the past, an SBR system was not feasible for home use due to complicated and expensive computer control systems. However, because of rapid development of affordable electronic technologies that provide advanced automatic control techniques, the SBR process becoming more feasible for individual home use.

A study conducted by members from Fujiclean Industry Co. Ltd., Toyohasi University of Technology, and Kyoto University showed impressive BOD_5, COD, soluble solids, total nitrogen, ammonia, and total phosphorus concentration reductions using an SBR. (Imura, 1980). The apparatus consisted of two 500 millimeter (1.64 ft.) diameter by 7 meters (22.75 ft.) long factory-reproduced-plastic (FRP) cylinders, each consisting of three chambers; a sequencing batch reactor, a disinfection chamber, and a sludge storage tank. The influent was derived from apartment houses at a flow rate of 20 cubic meters/day (5,279 GPD). Four batch cycles were repeated every day at 5 m^3 per day (1,320 GPD).

The SBR process typically consists of 4 individual steps. First, the influent is introduced into the reactor while an in-tank mixer and an aerator provides adequate mixing and addition of air. The second step is the activated sludge treatment (AS), which consists of mixing the influent by agitation, either

mechanically or with an aerator. By providing adequate oxygen for microorganism assimilation, bio-transformation occurs and suspended and dissolved organic material are sorbed onto cells and metabolized. The third step is the sedimentation process where solids settle under gravity. This is done by turning off any and all mixing devices in the chamber during the settling process. The fourth step consists of removing a portion of the sludge from the SBR chamber before the supernatant is discharged as effluent. Influx, anaerobic agitation, aerobic agitation, and sedimentation-discharge in this SBR study were automatically controlled.

Influent and treated water (effluent) were analyzed twice a month at the beginning and the end of the batch reaction process for pH, SS, alkalinity, COD, BOD_5, total nitrogen, ammonia nitrogen, nitrite and nitrate nitrogen, and total phosphorus. Table 4 summarizes the average influent and effluent values and ranges obtained from this SBR study.

Table 4: Average and Maximum/Minimum values for the SBR study

	Influent		Effluent		
Parameter	Average (mg/L)	Max-Min (mg/L)	Average (mg/L)	Max-Min (mg/L)	Average removal rate (%)
BOD_5	227	360 – 160	6.1	9.0 - 3.0	97.3
COD	95	190 – 45	9.5	13.0 - 7.7	90.0
SS	161	440 – 44	7.1	19.3 - 2.0	95.6
T-N	34	45 – 24	6.3	9.2 - 3.8	81.4
NH_4^+ - N	30	41 – 20	0.3	0.8 - 0.2	99.0
T-P	4.1	6.3 - 3.2	0.15	0.45 - 0.03	96.3

Results show that impressive effluent quality can be achieved regardless of changes in strength and temperature of influent and variations of MLSS in the

reactor. The diluted BOD_5 was used as the carbon source for the denitrification process. The SBR had complete nitrification as all ammonia was converted to nitrate. Phosphorus was also effectively removed during the anaerobic process. Phosphorus increased during the aerobic process and then decreased in the anaerobic process (Imura, 1980). This indicates that phosphorus was excessively absorbed into the cells during the anaerobic process and was then released during the AS process, attributing a removal rate of 96%.

Of course, the SBR process has its disadvantages too. Sensitivity to shock loads, additional storage requirements for overflow and surge protection, and sophisticated computer controlled technologies to control sequencing of the batch process to name a few. Depending on the level of sophistication and expectations, these factors may increase the cost for an individual homeowner to own and operate. Increased complexity in adding electronics, alarms, computer programming, controls, and electro-mechanical devices means an increase in maintenance and monitoring costs to a homeowner. If a malfunction occurs in an SBR, a significant decrease in effluent quality can be experienced or equipment can be damaged. Control and electronic monitoring can be overcome by affordable off-the-shelf technologies available today. Costs can be mitigated by distributing them over several homeowners using such a system and/or using creative financing plans, such as amortizing and allowing federal or state entities to subsidize. Nevertheless, if costs can be mitigated and

successful control and monitoring can be achieved through affordable technology, SBR offers very promising results.

CHAPTER 4
PROJECT SETUP AND TESTING

4.1 INTRODUCTION

National Sanitation Foundation (NSF) Standard-40 protocol was used to evaluate the WTP unit performance under standard operating conditions. To certify the UCZ-5, stress loading is also required under NSF-40; however, stress loading was not part of this validation study (it is being accomplished by others) and therefore will not be discussed. The data obtained at standard operating conditions will be combined with the future stress loading data as part of documentation to eventually certify the BEST's UCZ-5 WTP unit.

4.2 GENERAL SETUP

The BEST UCZ-5 was placed next to the influent channel at Sand Island Wastewater Treatment Plant (SIWWTP) to simulate raw influent experienced at a residential location. Photograph 1 is SIWWTP influent channel, which is used to supply wastewater to six individual advanced primary clarifiers and was tapped to supply raw wastewater to the tank. The tank was placed next to the influent channel (Photograph 2). Next to the tank is a storage shed that was used to house necessary equipment, materials, and instrumentation for the research project. The area above the storage shed was used to stage the influent control apparatus (bucket) and an ISCO 3700 sampler to collect the effluent samples. The equipment, instrumentation, and appurtenances will be

discussed in detail in the following sections. A sump pump operating continuously was connected to a 2 inch PVC pipe and submerged in SIWWTP's influent channel. To illustrate the size and type of sump pump used, Photograph 3 shows the old sump that was replaced with a new pump. A TEEL cast iron sump pump capable of operating at 1,750 RPM provided approximately 200 GPM of raw wastewater to the bucket apparatus located approximately 10 feet above the surface water line of the influent channel. Piping was connected to the TEEL sump pump (labeled A), to a valve that provides flow diversion for system maintenance (labeled B), and overflow protection for the influent control apparatus (labeled C), shown in Photographs 3, 4, and 5. These three pipes were inserted into the channel by detaching a small access plate covering the influent channel. Photograph 4 shows a direct overhead view of the disconnected access way and piping placed vertically into the influent channel. Photograph 5 shows the remaining piping network and where they connect. The influent pipe (labeled A) is connected to a "T" section. One side of the "T" section is connected to a manual gate valve, which is connected to piping (labeled B) that diverts the flow back to the influent channel, continuously to prevent sump-pump overload. This valve and piping section provides diversion in case system maintenance is required without stopping the sump pump. The manual valve and piping also provides an alternate way to collect influent samples without operating the WTP's instrumentation. The other side of the "T" section was reduced from 2 inches to 1 inch piping using a reducer. The

reduced section was connected to a manual PVC "T" valve and then plumbed into an electro-mechanical valve assembly before being inserted into the influent control apparatus. In case the influent control apparatus overflows, water will be diverted back to the influent channel by an overflow pipe inserted about one inch below the bucket's lip (label C). Photograph 6 shows a close up of the electro-mechanical valve used to control flow into the bucket, the PVC manual "T" valve, and gate valves used to isolate the system. The electromechanical valve is a ASAHI/American Electromni corrosion resistant thermoplastic uni-body ball valve manufactured with Teflon/EPDM dual seat and EPDM stem seal capable of a 5 second response time to open and close. This valve was connected to a Programmable Logic Control (PLC) unit located inside the storage shed which, when activated, allowed wastewater to flow into the influent control apparatus. Photograph 7 shows the influent control apparatus during a fill cycle. When the raw influent reaches approximately 6.25 gallons, a float switch located inside the bucket closes the circuit, the electromechanical valve closes and stops the raw wastewater from flowing into the bucket. Approximately one minute after the influent electromechanical valve closes, a second electromechanical valve (between the bucket apparatus and the WTP) opens allowing the collected wastewater to enter the WTP. The valve then closes and is ready for the next cycle to start. These two valves operated flawlessly during the 6 month testing duration. This cycle is repeated until the volume required at each time interval is met. Photographs 8 and 9 show respectively a longitudinal and close-up view of

the piping connecting the bucket to the WTP via the second electromechanical valve. After the wastewater passes through the WTP, the treated effluent is discharged back into the influent channel via two inch PVC pipe.

4.3 INFLUENT CONTROL AND SAMPLING SCHEDULE

Flow rates were automatically controlled using the influent control apparatus (bucket), a PLC unit containing a stored program, two automatic sprinkler timers, and the two electromechanical valves. The logic unit was activated electronically using two automatic sprinkler timers that were set at prescribed time intervals of 0600 hours, 1100 hours, and 1700 hours. Photograph 10 shows the logic unit, and the two automatic sprinkler timers attached to the inside panel of the storage shed (PLC unit is the box unit located above the two automatic sprinkler timers). The logic unit was a Mitsubishi Fxo-14MR-ES1UL series programmable controller (SN 645140). The logic unit was preprogrammed by Dr. Roger Babcock using a computer and MELSEC MEDOC programmable software to develop the Programmable Logic Controller (PLC) program. The MELSEC MEDOC PLC program can be programmed in instruction or ladder diagram format. The ladder diagram format was used to program and control the Mitsubishi Fxo logic unit for this study. Photograph 11 shows the computer used to upload or download the program to the PLC. A cable is connected from the computer to the PLC to upload the computer program. After the PLC accepts the code, the computer can then be turned off

indefinitely. The automatic sprinkler timer located directly below the logic unit is a Rainjet 6 zone, single program timer manufactured by Lawn Genie. This timer was used to activate the logic unit for the 6 o'clock cycle. A Rain Bird ISA 300/400 series timer (timer below the logic unit and the Rain Jet timer) having 6 pre-set schedules and three available start times for each station was used to activate the 11 and 5 o'clock cycles.

Approximately one minute after the PLC receives the electronic signal from one of the automatic sprinkler timers, the influent electromechanical valve opens and allows wastewater from the influent channel, being pumped by the sump pump, into the bucket apparatus located on top of the storage shed (Photographs 5, 7, and 8). The bucket was used to control the volume of wastewater into the WTP according to NSF-40's loading requirements outlined in Table 5. The bucket would fill and empty depending on which time cycle was activated for an average daily flow rate of 400 GPD. Four hundred GPD was chosen because it simulates a family of four using an average water rate of 100 gal/day/capita.

Table 5: Standard performance period loading conditions
(6-month duration at 7 days a week)

Time of day (hr)	Daily flow (%)	Volume (gal)	Bucket fills (#)
0600 - 0900	35	140	22
1100 - 1400	25	100	16
1700 - 2000	40	160	26

The UCZ-5 was designed to treat domestic wastewater for a family of five in Japan; however, a family of five in the Japan uses less water than a family of

five residing in the United States. The Japanese standard average per capita flow for domestic wastewater treatment is assumed as 0.25 m^3/day/person (66 gallons/day/person). Therefore, the standard loading rate in Japan for a family of five would be 1.25 m^3 /day (330 GPD) versus 1.5 m^3/day (400 GPD). On the basis of hydraulic loading rate, the UCZ-5's design capability was overloaded by 21%.

In Japan, the volume of a secondary treatment tank is calculated using the formula below:

$$V = 2.8 + 0.68(n-5)$$

where: n is the population for treatment
V is volume in cubic meters

For a family of five, the formula calculates a volume standard capacity of 2.8 m^3 (740 gallons); however, the UCZ-5 has been constructed with a total capacity of 3.02 m^3 (797 gallons) (Hozo, 1997). Evaluating on the basis of retention time for a family of five, the Japanese loading rates would have an HRT of 2.4 days (3.02 m^3/1.25 m^3 day) versus 2.0 days (3.02 m^3/1.5 m^3 day). Part of validating this tank was to determine if the tank satisfies NSF 40's effluent quality requirements with the UCZ-5 design capacity being over stressed by 21%.

To assess the basic performance of the residential wastewater treatment unit, a protocol for sample collection (see Table 6) was developed to meet the NSF Standard 40 requirement for "composite" samples.

Table 6: NSF Standard 40 Collection Requirements

Type	Time (hrs)	Amount	Composite*
Morning	0630, 0730, 0830	300 ml each	350 ml
Afternoon	1130, 1230, 1330	300 ml each	250 ml
Evening	1730, 1830, 1930	300 ml each	400 ml
			1000 ml

An automatic sampling device (ISCO 3700) was used to collect the morning, afternoon, and the evening samples for influent and effluent according to the schedule shown in Table 6. Photograph 12 shows the ISCO 3700 automatic sampler used to collect the influent samples. The composite samples were produced by taking the amount prescribed (350, 250, and 400 ml) and mixing them appropriately creating 1000 ml bottle of mixed influent and effluent samples. These volumes correspond to 35% of the daily flow in the morning, 25% at noon, and 40% at dinner time. Manual grab samples were also obtained from the aeration section of the tank using two 1000-ml ISCO portable bottles. According to NSF standard 40 - section 5, samples are required to be collected and analyzed on a five-day per week basis for a minimum of 6 months in order to be eligible for qualification by the State Department of Health (SDOH). These samples were collected Monday through Friday.

4.4 MEASUREMENTS AND METHODS

The ultimate goal of testing "The Tank" was to determine whether this system could produce effluent that satisfied the EPA's secondary treatment

guidelines (therefore receiving a class 1 status). Table 7 summarizes the analyses performed:

Table 7: NSF Standard 40 Testing Protocol

Sample Location	Type of Sample	Frequency of test	DO mg/L	BOD_5 mg/L	Total Susp. Solids mg/L	Volatile Susp. Solids %	Settleable Solids ml/L (30 min)	Temp °C	pH
Raw Influent	24 hr Composite	Daily (M-F)		X	X	X		X	X
Final Effluent	24 hr Composite	Daily (M-F)	X	X	X	X		X	X
Aeration Tank	Grab	Daily (M-F)	X		X	X	X	X	X

Samples were measured as per the table above according to procedures described in Standard Methods (Standard Methods, 1995). In addition to these daily tests, NSF Standard 40 requires that the effluent be diluted by 1000 times and tested at least three times during the six-month evaluation for color, odor, oily film, and foam.

Testing of Ammonia, Nitrate, Nitrite, Organic Nitrogen, Phosphorous, Turbidity, and Oil and Grease were conducted at various times during the testing period to study nutrient removal capability. Soluble Biological Oxygen Demand (SBOD) of each chamber was also measured at steady state conditions for a three week period (7 December to 25 December 1998). These tests are not required by NSF-40.

4.4.1 ON SITE MEASUREMENTS

On site measurements consisted of measuring the concentration of Dissolved Oxygen (DO), pH, and temperature in the aerobic section of the tank. These parameters are important indicators for healthy microorganisms for attached and suspended activated sludge treatment. DO and pH was measured using Standard Methods 4500 O G and 4500 H, respectively. Temperature was measured using the thermistor integral to the Dissolved Oxygen probe. The DO and pH meter were calibrated on site before being used.

4.4.2 SOLIDS (TSS, VSS, AND SETTLEABLE)

Measurement of seattleable solids (SS) is important in wastewater treatment system design; specifically primary sedimentation and settling tank design. After primary settling, the amount of Total Suspended Solids (TSS) (fixed and volatile) is used to determine the expected loading for secondary biological treatment and to control aeration requirements during the activated-sludge process. Volatile Suspended Solids (VSS) is used to estimate the amount of organic material a biological treatment unit will have to process and control.

Standard Methods 2540 F (Standard Methods, 1995) was used to measure SS in mL/L. SS was determined by placing 1 liter of well mixed sample into a standard Imhoff cone. The solid matter was allowed to settle for 45 minutes, any matter retained on the side of the cone was gently released using a stir bar, then

after 15 minutes the amount of seattleable solids was read from a graduated scale etched on the side of the Imhoff cone.

Standard Methods 2540 D was used to measure TSS. TSS is determined by the amount of solids retained on a specified filter. The water sample was poured through a Whatman grade 934 AH filter that was pre-cleaned dried, and weighed. The filter was washed using deionized (DI) water then placed in an oven at 103 °C for approximately 24 hours. The filter was assumed to be clean and completely without moisture and ready to be used. After filtering the sample, it was placed into an oven at 103 °C for approximately another 24 hours and then placed in a dessicator to cool. After the sample cooled to ambient temperature, the sample was weighed. TSS was determined by the difference in weight before and after the sample was applied and units converted appropriately. Conveniently, this sample can be further tested to determine VSS by placing the sample into an oven at 550 °C to volatilize the organic material (Standard Method 2540 E). The difference in weight between the sample weight after volatilizing and TSS dried weight of the sample is VSS measured in mg/L. The difference between the weight to determine VSS and TSS is the amount of fixed suspended solids contained in the sample.

4.4.3 BIOCHEMICAL OXYGEN DEMAND (BOD)

Biochemical Oxygen Demand (BOD) is a measure of the oxygen consumed by microorganisms during the biodegradation of organic wastes under

aerobic conditions. The BOD test is one of the most widely used parameters to determine the pollution strength of domestic and industrial wastewater. Theoretically, an infinite time is required for complete biological oxidation of organic manner, but a standard method of 5 days at 20°C has been chosen to compare data under different geographical areas and their environmental conditions. The 5-day BOD test is annotated as BOD_5. The 5-day BOD has been found to be about 70 to 80 percent of the ultimate BOD (BOD_u). BOD is an important parameter that is used to size biological treatment facilities and to measure the efficiency of operations in the treatment of wastewater. All samples were measured using Standard Method 5210 B. An YSI Model 58 Dissolved Oxygen meter attached to an oxygen probe with an agitator was used to measure all samples.

4.4.4 TURBIDITY

Turbidity is a measure of light-transmitting property of the water with respect to colloidal and residual suspended matter. It is essentially the measure of water clarity. Turbidity is based on the comparison of the intensity of light scattered by a sample as compared to the light scattered by a reference suspension under the same conditions (Metcalf and Eddy, 1991). This relative measurement called Nephelometric Turbidity Units (NTU) ranges from 0.05 to 40 NTU and is read directly from the instrument.

There is a relationship between turbidity and suspended solids for settled secondary effluent form the activated sludge process:

Suspended Solids, SS, mg/L = (2.3 to 2.4) X (turbidity, NTU)

Standard Method 2130 B was used to measure turbidity. The Hach Turbidimeter (model number 2100A) was used to measure all samples.

4.4.5 AMMONIA

The method used for this research project was 4500-NH_3 D, Ammonia Electrode Method. The pH of the sample was raised to 11 or above using 10N NaOH. The ammonia was converted to NH_3 gas, which passes through a membrane probe detecting a change in pH, which was set against a curve with known standards. The standards developed were serial dilutions of 100, 10, 1, and 0.1 mg/L of NH_3-N. The ammonia probe used was the Orion Model No. 95-12 Ammonia Probe and the readings were measured with the Orion Model 720A Meter.

4.4.6 TOTAL KJELDAHL NITROGEN

Total Kjeldahl Nitrogen (TKN) is the total amount of nitrogen in the form of organic and ammonia nitrogen. Since the ammonia concentration was measured separately, the TKN value was used to determine the amount of organic nitrogen present. The source of organic nitrogen in wastewater is mainly from human wastes. Most of the organic nitrogen in wastewater is in the solid form and a majority of it can be settled out in a primary settling chamber.

The method used to measure TKN was Standard Method 4500-N_{org} C. Semi-Micro-Kjedahl Method. After the TKN was hydrolyzed and digested using an oxidizer to form ammonia, an ammonia probe was used to measure TKN according to Standard Methods 4500-NH_3 D. Ammonia-Selective Electrode Method (Standard Methods, 1995). A blank sample and ammonia standards were used to calibrate the Orion 720A meter. Samples were spiked using varying amount of samples being tested and standard ammonia concentrations to validate testing.

4.4.7 NITRITE/NITRATE

Nitrate was determined using Method 4500-NO_3 E, Cadmium Reduction combined with Method 4500-NO_2^-B, Colorimetric Method (Standard Methods, 1995) to measure absorption. The samples were filtered and then passed through a column containing copper coated cadmium particles used to convert nitrate to nitrite. Measuring known concentrations of nitrate using the spectrophotometer set at 543 manometers (nm), a curve of nitrate reduced to nitrite was constructed. The slope of the curve was multiplied by the adsorbance of each sampled tested to obtain nitrate present in each sample. Nitrate was then determined by the difference between reduced samples (containing nitrate and nitrite ions) and the unreduced samples (containing nitrite ions).

The reduced samples were then analyzed for nitrite present using a spectrophotometer set at a wavelength of 507 nm. By using the slope of this curve, the amount of reduced nitrite was calculated for each sample. Method

4500- NO_2^-B, Colorimetric Method (Standard Methods, 1995) was also used to measure NO_2^- concentration in each sample. It was found that the concentrations of nitrite and nitrate were very low (below 1.0 mg/L-N). To facilitate measuring numerous samples, it was decided to evaluate nitrate as nitrate/nitrate-N to calculate other forms of nitrogen.

4.4.8 PHOSPHORUS

The standard method selected to measure Total Phosphorus and Orthophosphorus was Method 4500-P C, Vanadomolydophospheric Acid Colorimetric Method. To measure total phosphorus in the sample, it was necessary to hydrolyze and digest 50 mL of sample using sulfuric acid solution and an oxidizer with the aid of an autoclave. Standard Phosphate solutions and a blank sample were used to develop standard curves for each test that could be used to determine phosphorus concentrations. Solutions were also spiked using standards and samples to validate testing. The variance of the standard curves for digested and undigested samples were all approximately 1.00, showing good accuracy in sampling. Orthophosphate was determined using the same procedure as total phosphorus, except hydrolyzing and digesting of each sample was omitted. A colorimetric test at a wavelength of 470 nm was used to determine the concentrations of total and reactive phosphorus.

4.4.9 OIL AND GREASE

Standard method 5520 B, Partition-Gravimetric Method was used to measure oils and grease in the influent and in the effluent. Standard methods list Freon 13 as the solvent to use. However, because of environmental concerns associated with CFCs (destruction of the stratospheric ozone layer), Hexane has been substituted in the method as an acceptable solvent. The method involves using n-Hexane to extract oil and grease from the water. The solvent containing oil and grease substance was decanted into a separate container for distilling and weighing after the procedure was repeated three times, increasing the recovery rate of the experiment. The solvent, which has a lower boiling point than water, is vaporized between 85 °C and 95 °C using a Rota-vacuum apparatus (Rotavapor Model RE 121). What remains is the oil and grease present in the sample. The sample within the bottle is placed in a dessicator to adsorb any residual water, then weighed. The difference between the final weight of the dried sample and bottle and the initial bottle weight divided by the original volume is the concentration of oil and grease contained in the sample, measured in milligrams per liter.

4.5 DESCRIPTION OF UNIT

Description of the UCZ-5 (The Tank) was briefly discussed in section 2.2. This section provides a more detailed description of each chamber integral to the tank in sequence.

4.5.1 ANAEROBIC CHAMBERS

The raw wastewater from the influent control apparatus flows into the first of two anaerobic chambers under gravity. The UCZ-5 would be installed below ground level with access ports at ground level if and when installed for residential use. A negative gradient between the collection pipe exiting the home and the tank's entrance would produce natural flow to the tank, similar to the experimental set-up. Photograph 13 shows an overhead view of anaerobic chamber #1. The first anaerobic tank has a volume of 0.84 m^3 (220.9 gal.). At 400 GPD, this volume provides an average detention time of 13 hours (Equation (1), Appendix A). The first anaerobic tank acts as a primary clarifier and anaerobically biodegrades organic matter into stable products. Chamber #1 contains filter media to promote anaerobic attached biological treatment of the influent that reduces sludge volume when compared to the aerobic process. The filter media within chamber #1 are spherically shaped media that provides good split-flow diffusion. The filter media is supported by plastic mesh suspended a few inches above the bottom of the tank and is held from the top by plastic meshing attached approximately half the distance of the tank's height. Exact dimensions of filter media are proprietary information.

Photograph 14 shows access to anaerobic chamber #2 (left half of access port) and the aerobic chamber (right half of access port). The volume of anaerobic chamber #2 is 0.7 m^3 (184.7 gal.). Chamber #2 also treats the

wastewater anaerobically using attached growth process. Filter media contained in chamber #2 is different than in chamber #1 in that the filter media are cylindrically shaped. Chamber #2 also provides denitrification of nitrified recycled wastewater extracted from the bottom of the adjacent aerobic chamber. At 400 GPD, chamber # 2 provides a hydraulic retention time of approximately 11 hours before proceeding to the adjacent aerobic chamber via a vertical flow channel (Equation (1), Appendix A). The total anaerobic HRT between both chambers is approximately 24 hours at 400 GPD.

4.5.2 AEROBIC CHAMBER

Photograph 14 shows an overhead view of the aerobic chamber with associated control valves (right half of access port). The purpose of the aerobic chamber is to provide biodegradation of organic matter using aerobic bacteria. Photograph 15 a close-up of the chamber, shows ample amount of air providing satisfactory oxygen diffusion. The volume of the aerobic chamber is 1.03 m^3 (274.0 gal.). At 400 GPD, the aerobic tank provides an HRT of approximately 16 hours (Equation (1), Appendix A). A plastic hose connected to a blower housed in the storage shed, Photograph 16, transports the air. The blower operates on 115V/60Hz power, requires 61 watts to supply air at a rate of 60 liters/hour (0.035 cfm) at 0.2 kpf/cm^2 (2.8 PSI). Yasunaga Corporation (Model LP 60A) manufactured the blower. The valves connected to the manifold are used to back flush the aerobic chamber (red valves), adjust air flow to the aerobic

chamber (blue valves), control air to the sludge return line (gray valve), and provides air release (yellow valve). Refer to the Operation and Maintenance Manual for detailed instructions on how to use the valves in conducting associated maintenance to the tank. The aerobic chamber also contains a layered array of corrugated plastic media that supports aerobic attached-growth treatment, as well as, suspended-growth treatment above and within the filter media.

4.5.3 SETTLING CHAMBER

Following aerobic treatment, the wastewater flows into a sedimentation chamber via a rectangular access port. Photograph 17 is a digital picture showing the aerobic chamber (right), settling chamber (left), recycle flow control box, and the chlorine canister, contained within a third access port. The settling chamber has a volume of 0.42 m^3 (111.2 gal.), providing a detention time of 6.7 hours at 400 GPD (Equation (1), Appendix A). The settling chamber provides separation of flocculated organic particles following the aerobic process. If used, the chlorine canister is held in place by a plastic tube, which is integral to a weir bridge attached to the tank, Photograph 17. The aerobically treated water flows past a weir where water is directed into the feed tube using a channel. The clear plastic canister is slotted at the bottom, exposing tablets to the flow. Chlorine tablets are inserted into the clear plastic canister and are dissolved as water

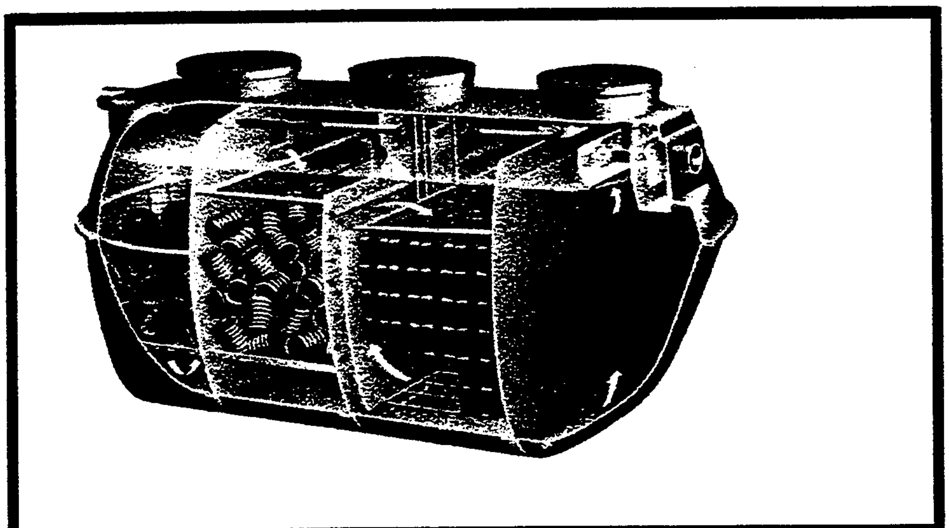
comes in contact through the slots (Photograph 18). The water flows into a chlorine contact basin until being displaced by inflow.

4.5.4 RECYCLE LINE

A 2 inch recycle line was constructed into the UCZ-5 to provide denitrification of wastewater that has been nitrified in the aerobic chamber. Exfoliated sludge from the aerobic process is extracted from the bottom of the aerobic chamber, is aerated in the recycle flow control box (Photograph 17), and then recycled into anaerobic chamber #2. Photographs 19 and 20 show the recycled water being discharged into anaerobic chamber #2.

4.5.5 SUMMARY

The relative size and shape with respect to filter media and chamber volume can be seen in following diagram.



(Small-Scale Combined Collection Treatment Unit by BEST Industries, May 1996)
Figure 1: BEST UCZ-5 Tank Diagram

Overall UCZ-5 tank dimensions are 1.32 m wide by 2.45 m long by 1.77 m tall (4.3 ft wide by 8 ft long by 5.6 ft tall). Table 8 summarizes internal volumes of the UCZ-5 unit in metric and SAE units.

Table 8: UCZ-5 Tank Specifications (400 gal/d capacity)

Tank Compartment	Volume (cubic meters)	Volume (cubic feet)	Volume (gal)
Anaerobic Chamber #1	0.84	29.52	220.85
Anaerobic Chamber #2	0.70	24.69	184.66
Aerobic Chamber	1.03	36.62	273.95
Settling Tank	0.42	14.87	111.22
Disinfection Tank	0.02	0.78	5.81
Total Volume	3.02	106.50	796.48

CHAPTER 5
EXPERIMENTAL RESULTS AND DISCUSSION

5.1 NSF-40 CRITERIA

One of the objectives of this project was to evaluate the performance of the manufacturer's supplied portable wastewater treatment tank (BEST UCZ-5) using the NSF-40 testing protocol and at the same time validating the manufacture's specifications. This project addressed the protocol's so-called "standard performance test", which is required to be at least 6 months in duration. The expected results are that the effluent quality should meet or exceed Class I NSF Standard 40 requirements (see table 9 below).

Table 9: Expected Effluent Water Quality Results

Parameter	Limit
BOD_5	< or = 30 mg/L & > or = 85% removal. (Arithmetic mean over 30 days) < or = 45 mg/L (Arithmetic mean over 7 consecutive days)
TSS	Same as above
pH	Between 6 and 9
Odor	Non-offensive
Oily Film	Non-visible other than air bubbles
Foam	None
Color	< 15 units

5.2 MEASUREMENTS

Five-day Biological Oxygen Demand (BOD_5), Seattleable Solids (SS), Total Suspended Solids (TSS), Volatile Suspended Solids (VSS), pH, temperature, and Dissolved Oxygen (DO) were measured 5 days a week (M - F)

for a total of 6 months (September 23, 1998 through March 26, 1999). All analysis were performed at the University of Hawai`i's Environmental Laboratory, except for pH, temperature, and dissolved oxygen, which were measured on site during sample collection between the hours of 9:00 A.M. and 11:00 A.M. These measurements were taken in the tank's aerobic chamber. Influent, aerobic, and effluent samples were measured for BOD_5, TSS, and VSS for the six-month steady state period. Soluble Biological Oxygen Demand (SBOD) was also measured at each chamber for a 26-day period (December 7, 1998 through February 25, 1999). BOD_5, TSS, VSS were also measured in each chamber from December 30, 1998 through February 3, 1999.

5.3 IN-SITU MEASUREMENTS

In-situ measurements consisted of measuring DO concentration and temperature using a portable meter with an attached probe and measuring pH using an Orion 720A meter and pH probe. The DO meter was calibrated at saturation and corrected for water temperature. The pH meter was calibrated using standards available on-site. In-situ measurements were taken while collecting the samples for laboratory analysis.

Table 10 summarizes the DO, pH, and temperature measurements for the 6 month standard performance test period.

Table 10: Summary of In-situ Measurements

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Dissolved Oxygen (mg/L)	0	6.5	3.4	1.3
pH	6.8	7.6	7.4	0.2
Temperature (°C)	22.0	27.5	25.4	1.3

5.3.1 DISSOLVED OXYGEN

Figure 2 shows a plot of dissolved oxygen taken from September 23, 1998 through March 23, 1999, during the 6-month standard performance test period. DO was measured above 5 mg/L at the start of the test period, which was about two weeks after initial start-up. The average DO was 3.4 mg/L with a standard deviation of 1.3 mg/L of O_2. NSF-40 does not have a requirement for DO content.

The DO fell below 5.0 mg/L about two weeks after the measurements started or about 4 weeks after the tank was put on-line. The trend line shown on Figure 2 shows a negative slope or decrease in dissolved oxygen in the aerobic chamber as a function of time. The slight increase of DO in the beginning of the test cycle and the decrease is suspected to be caused by two reasons. First, the DO was probably above 5.0 until the microorganisms acclimated to the system and the waste began to fully utilize the available DO in their assimilation of organic matter in the wastewater. This usually takes about 1 month. Secondly, the buildup of biomass on the plastic media continues until respiration

requirements surpasses the amount of DO the blower can supply to the aerobic chamber, approximately 6 months after startup. Also, as temperature decreases, the ability of oxygen to dissolve in water increases. In November and January there was a 1°C incremental drop in temperature. This drop in temperature seemed to help increase DO. By analyzing the plot, the DO measured in the chamber seemed to slightly increase when the temperature decreased for both of these months. After the second month, the DO slightly increased, then decreased until the DO fell below 1.0 mg/L. The Tank was taken off line on March 12, 1999 for cleaning as per manufacturer's recommendation. The DO quickly recovered the next day and started to increase, approaching the 5.0 mg/L mark. The decrease in DO was caused by the accumulation of attached biomass on the aerobic filter media requiring more oxygen than the blower could provide. This was indicated by effluent BOD_5 remaining below 30 mg/L, even though the DO decreased below 1 mg/L. Periodic back flushing of the aerobic chamber and cleaning of air diffusers (every 3-6 months) should prevent the biomass from accumulating to a level that cause DO to decrease below the recommended level.

The trend-line closely approximates when cleaning of the tank can be anticipated, which is recommended when the DO decreases below 1.0 mg/L. Cleaning the tank was accomplished by completely removing the sludge in anaerobic chamber #1 and about ½ of the sludge in anaerobic #2. A sump pump was used to remove the sludge from both anaerobic chambers by

submerging the pump through the vertical access channels. When the filter media was exposed by partially dewatering each chamber, the biomass was cleaned with a water hose and also removed by the sump pump.

The data indicate that the tank requires cleaning about every 6 months. Periodic back flushing as per the Operations and Maintenance instructions may extend this time.

5.3.2 pH

Figure 2 shows pH measured during the test period. pH was very stable, averaging 7.4 with a standard deviation of 0.2. NSF-40 requires that pH remains between 6.0 and 9.0 (NSF-40, 1975). The UCZ-5 met easily met this criteria.

5.3.3 TEMPERATURE

Figure 2 plots temperature as a function of time. Average temperature for the 6 month test period was 25.4 °C with a standard deviation of 1.3 °C. As discussed in section 5.3.1, the temperature decreased one degree°C in November 1998 and another degree in January 1999. The temperature changes very little in this geographic area (Hawai`i), so temperature changes for biological processes are not a large concern as in other parts of the United States. However, it is important to maintain the temperature between 20°C and 50 °C for mesopholic aerobic treatment of wastewater.

5.4 TOTAL AND VOLATILE SUPSENDED SOLIDS

Figures 3 and 4 show TSS and VSS concentrations during the 6-month standard performance test period. The average influent and effluent TSS concentration were 128.0 mg/L and 13.1 mg/L with a standard deviations of 27.6 mg/L and 6.9 mg/L, respectively. The average influent TSS concentration closely correlate with published values experienced at SIWWTP (116 mg/L through 126.5 mg/L). The average influent and effluent VSS concentrations were 109.1 mg/L and 10.1 mg/L with standard deviations of 22.6 mg/L and 4.7 mg/L, respectively. Despite the typical fluctuations in influent TSS and VSS, the effluent TSS and VSS were relatively stable. VSS was between 80 and 85% of TSS values. VSS influent values were 85% of influent TSS values and effluent VSS values were 80% of effluent TSS values. The average removal rate for TSS was 91.8% and 92.7% for VSS. The NSF-40 criteria for suspended solids is that the arithmetic mean of all effluent samples collected in any period of 30 consecutive days shall be less or equal to 30 mg/L, the arithmetic mean of all effluent samples collected in any 7 consecutive days is less than or equal to 45 mg/L, and removal must be greater or equal to 85% (NSF-40, 1984). The UCZ-5 easily met all three criteria. A modification was made to the recycle line on November 30, 1998 that caused the effluent TSS and VSS to increase to 33 mg/L and 22 mg/L; respectively, before stabilizing below 30 mg/L the next day. It was noticed that when the DO in the aerobic tank fell below 1.0 mg/L, the TSS

and VSS remained below 30 mg/L. The anaerobic filter process was suspect in removing suspended solids below 30 mg/L before entering the aerobic chamber.

Figures 5 and 6 show a plot of average, minimum and maximum values for TSS and VSS as a function of chamber location for samples taken from December 30, 1998 through January 29, 1999. Twenty-three samples for each chamber location were collected 5 days per week for this 1 month test period. There were 34 recycle samples collected 5 days a week (Monday through Friday) over a 1 ½ month period. The influent and effluent were composite samples taken from the automatic samplers. The values for the anaerobic chambers, aerobic chamber, and the recycle were measured using grab samples. These two graphs show treatment of TSS and VSS with respect to tank location, as the wastewater flows from one section to another. The upper data points (diamond shapes) and the lower data points (dots) represent the maximum and minimum values measured during the sampling period. Figures 5 and 6 show that TSS and VSS are reduced by 68% and 75%, respectively by anaerobic chamber #1. Anaerobic chamber #2 reduces TSS and VSS by an average 60% and 63%, respectively, and the aerobic chamber reduced TSS and VSS by an average 29% and 24%, respectively. The anaerobic chambers provide significant TSS and VSS reductions to the entire purification process. Tables 11 and 12 summarize TSS and VSS values, respectively. The highest value for TSS in anaerobic chamber #2 was 24 mg/L. This confirms that the attached growth process in the anaerobic filter reduces suspended solids below

30 mg/L before reaching the aerobic process. Tables 11 and 12 show that the average TSS and VSS concentrations for the 1 month chamber analysis are 17 mg/L and 10 mg/L with standard deviations of 4 mg/L and 3 mg/L, respectively. The anaerobic chambers provide significant suspended solid reductions of 87% for TSS and 90% for VSS before reaching the aerobic process.

Table 11: Summary of TSS values for Chamber Analysis

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	99	161	134	16
Anaerobic #1	13	76	43	15
Anaerobic #2	3	24	17	5
Aerobic Chamber	6	24	12	4
Effluent	7	23	15	3
Recycle	2	18	12	4

Comments: Influent, Aerobic, and Effluent samples were composite and aerobic samples were grab samples taken from 12/30/98 through 1/29/99. Recycle samples were taken from 12/14/98 through 1/29/99.

Table 12: Summary of VSS values for Chamber Analysis

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	80	129	108	13
Anaerobic #1	13	55	27	10
Anaerobic #2	4	15	10	3
Aerobic Chamber	5	17	8	3
Effluent	6	20	10	3
Recycle	2	12	8	2

Comments: Influent, Aerobic, and Effluent samples were composite and aerobic samples were grab samples taken from 12/30/98 through 1/29/99. Recycle samples were taken from 12/14/98 through 1/29/99.

5.5 SETTLEABLE SOLIDS

Figure 7 represents the concentration of settleable solids in mL/L in the aerobic chamber for the 6 month standard testing period. The maximum settleable solids experienced by the tank was approximately 1.1 ml of solids with an average of 0.1 mg/l and a standard deviation 0.2 mL/L. Settleable solids represent solid material that would settle under gravity in a primary clarifier. Concentrations seemed random and probably represents biomass or flocculated material that has "sloughed off" the plastic filter media in the aerobic chamber. This happens when the biomass reaches a thickness where it cannot be supported or due shearing from the abrasive action caused by the air diffusers. Large peaks in settleable solids seem to occur at 1 month intervals and during periods surrounding holidays or events that could produce high solids loading (Thanksgiving, Christmas, New Years, Super bowl, Spring Break, etc.). With an average value of 0.1 mL/L over 6 months, the settleable solids concentration is considered negligible.

5.6 BIOCHEMICAL OXYGEN DEMAND

Figure 8 plots the influent and effluent BOD_5 for the 6-month testing period. NSF-40's requirement for BOD is the same as that for suspended solids. The average BOD_5 for the influent and effluent were 146.4 mg/l and 13.9 mg/l with standard deviations of 28.0 mg/l and 6.0 mg/l, respectively. The influent BOD_5 was within the published SIWWTP BOD_5 values of 138 to 150 mg/L. The

effluent BOD_5 achieves NSF 40's criteria of 30 mg/l for Class I effluent. Similar to TSS and VSS, the effluent BOD remained fairly stable despite the large variations in the influent BOD_5. The average BOD_5 removal rate for the six-month standard test period was 91%. Effluent BOD_5 increased above 30 mg/L from October 5, 1998 through October 8, 1998. This increase is believed to be caused by the microorganisms acclimating to their environment before stabilizing over the remainder of the 6 month test period. The BOD_5 approached, but did not reach 30 mg/L, when DO decreased below 1 mg/L. After performing the recommended maintenance on the tank, effluent BOD_5 decreased and stabilized to approximately 15 mg/L.

Twenty five samples were collected 5 days a week (Monday through Friday) from December 30, 1998 through January 29, 1999 to evaluate the average BOD_5 reductions performed by each chamber. A grab sample was collected from each spill over duct for a period of one month for the anaerobic chambers, aerobic chamber, and for the recycle. The influent and effluent samples were collected as composite samples as described in Section 4.3. Figure 9 represents the results of this analysis. The upper limit represented by triangles is the maximum value for each section of the tank. The data points marked as circles represent the minimum values measured for each section of the tank during the test period. The average is represented by a dash. The average influent BOD_5 for this period was 132 mg/L, which is within the standard deviation of the 6-month period (118 mg/L to 174 mg/L). Figure 9 shows that

anaerobic chamber #1 reduces BOD_5 by about 28%, chamber #2 greatly reduces BOD_5 by 84%, and the aerobic chamber reduces BOD_5 by another 17%. The reason for this is probably that the organic matter in the influent was reduced in anaerobic chambers #1 and #2 in the form of suspended solids such that the aerobic chamber did not have much organic material for additional biodegradation. The BOD concentration in the aerobic section is essentially the same as the effluent. Table 13 summarizes the BOD testing.

Table 13: Summary of BOD values

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	80	242	146	28
Anaerobic #1	57	138	105	20
Anaerobic #2	9	39	17	7
Aerobic Chamber	4	28	14	5
Effluent	6	37	13	6
Recycle	6	19	12	4

Anaerobic chamber #1 reduces settleable and suspended solids more so than anaerobic #2 while anaerobic chamber #2 reduces BOD_5 more so than anaerobic #1. This enables the aerobic process to effectively reduce BOD_5 levels below 15 mg/L on average. The aerobic, effluent, and recycle BOD_5 values were approximately the same. The aerobic chamber at this location (following the anaerobic chambers) polishes BOD causing material from the wastewater, but primarily is used to convert organic and ammonia-nitrogen to nitrite and nitrate through nitrification. The recycle supplies wastewater that has

been nitrified to anaerobic chamber #2. The anoxic condition of chamber #2 is designed to convert nitrate to gaseous nitrogen by the denitrification process (NO_3^- to N_2).

Using equation (2) (Appendix A), the surface area of the spherical media contained in chamber #1 was estimated to be 25 m^2 (270 ft^2). The volume of each chamber was given by the manufacturer (Table 8), surface area of each cylinder was directly measured, the sphere diameter is 15 cm. (5.9 in), and the percent of volume occupied by the plastic media and the packing factor was estimated to be 40% and 70%, respectively. The surface area of the cylindrical plastic media contained in anaerobic chamber #2 was estimated to be 37 m^2 (396 ft^2) using equation (3), Appendix A. The diameter of the string used to fabricate the cylindrical meshed structure was measured at 2 mm (0.078 in) and the length was estimated to be 36.6 m (120 ft). The cylindrical mesh structure is 12 cm (4.7 in) in diameter and 13.5 cm (5.3 in) in height. The packing factor and the volume occupied by the cylindrical plastic media was estimated to be 70% and 50%, respectively. The media located in the aerobic chamber could not be measured, but was estimated to have a specific surface area of plastic media used for a conventional trickling filter. Multiplying the specific surface area of 30 ft^2/ft^3 by the manufacturer's specified tank volume (36.6 ft^3) and considering approximately 80% of the tank is occupied by the aerobic filter media, yielded an aerobic media surface area of 880 ft^2.

Table 14 summarizes BOD removal rates based on volumes and media surface areas for each chamber (Equations (4) and (5), Appendix A). Average BOD_5 values (Table 13) and an average flow rate of 400 GPD were used to calculate the removal rates.

Table 14: Summary of BOD removal rates based on tank volume (mass/$10^3 L^3$•day) and media surface area (mass BOD/10^3 L^2•day):

Chambers	$\frac{Kg}{10^3\ m^3 \bullet day}$	$\frac{Lb}{10^3\ ft^3 \bullet day}$	$\frac{Kg}{10^3\ m^2 \bullet day}$	$\frac{Lb}{10^3\ ft^2 \bullet day}$
Anaerobic #1	73.8	4.6	2.5	0.5
Anaerobic #2	190.0	11.9	3.7	0.7
Aerobic chamber	4.4	0.3	0.05	0.01

Typical total BOD_5 media removal rates for biological contactors are 2.0 - 3.5 lb $BOD_5/10^3$ ft^2•day for secondary and 1.5 - 3.0 lb $BOD_5/10^3$ ft^2•day for combined nitrification treatment (Metcalf and Eddy, 1991).

Soluble Biochemical Oxygen Demand (SBOD) was also analyzed as a function of tank location. SBOD represents the amount of oxygen used by microorganisms to biodegrade soluble organic matter that can pass through a glass fiber filter with a nominal pore size of 1.2 micrometers. Since some of the suspended solids (including organics) have been removed by filtration, SBOD values should be less than BOD values. Figure 10 represent the results of SBOD sampled from 7 December through 25 December 1998. A total of 14 samples, 5 days a week (Monday through Friday) were collected within each

chamber. The plot shows that SBOD actually increases from an average 58 mg/l to 86 mg/l after anaerobic chamber #1 and then decreases to approximately 15 mg/l after anaerobic chamber #2. The overall SBOD reduction was 88%. SBOD is basically constant from the aerobic chamber to effluent and recycle, as expected. SBOD is expected to increase in anaerobic #1 because the particulate organic material in the wastewater was solublized under anaerobic conditions. Because most of the organic material is removed after anaerobic chamber #2, SBOD is approximately the same as the BOD concentrations for aerobic effluent samples. Table 15 summarizes SBOD results.

Table 15: Summary of SBOD values

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	44	72	58	8
Anaerobic #1	69	103	86	12
Anaerobic #2	9	28	15	6
Aerobic Chamber	5	16	10	4
Effluent	3	11	7	3
Recycle	5	12	8	3

5.7 TURBIDITY

Figure 11 shows the results of turbidity for unfiltered influent and effluent grab samples taken at random intervals. Thirty two analyses were conducted over the 6 month time period to establish a reasonable average and standard deviation. The average influent and effluent turbidities were 54.8 NTU and 6.3 NTU with a

standard deviation of 13.6 NTU and 5.0 NTU, respectively. As with the BOD data and TSS/VSS data, the influent turbidity was also highly variable, shown by the large standard deviation of 13.6 NTU. The minimum and maximum values for influent were 32 NTU and 84 NTU and 1.4 NTU and 30 NTU for effluent. As expected, the effluent behaved similarly to the suspended solid and BOD_5 data; effluent turbidity was fairly stable. The relationship between turbidity and suspended solids discussed in Section 4.4.4. was observed. Based on average values, the ratio of TSS/Turbidity for influent was 2.3 and 2.1 for effluent.

5.8 NITROGEN

Figures 12 through 15 represent results of Total Kjeldahl Nitrogen (TKN), Ammonia, Nitrite/Nitrate, and total nitrogen (TN) concentrations as function tank location; specifically, composite influent sample, grab sample from the aerobic chamber, and composite effluent sample, respectively. TKN is a measure of organic plus ammonia-nitrogen concentration in a sample. When ammonia is measured and subtracted from the TKN, the remainder is the amount of organic-nitrogen in a given sample.

Fourteen separate samples were collected to measure TKN (February 8, 1999, February 23 through March 4, 1999 (M-F), and March 18 through March 26, 1999). Figure 12 shows that TKN decreased from the influent to the aerobic section, then increased slightly. The decrease is probably due to ammonification of organic nitrogen in the anaerobic chambers followed by nitrification in the aerobic chamber. The increase from the aerobic chamber to

effluent is mostly likely not real, but rather a measurement artifact. It is difficult to determine the cause or amount of organic nitrogen removal before the aerobic section because a nitrogen analysis was not performed for each chamber. An increase of approximately 3 mg/L of ammonia-nitrogen was experienced between the influent and effluent (Figure 13). This is not usually expected. It could be because a large amount of ammonia was produced anaerobically due to ammonification and nitrification of the ammonia did not keep pace. TKN and ammonia-nitrogen concentrations essentially did not change between the aerobic section and the effluent. The average organic nitrogen present in the influent, aerobic grab sample, and effluent were 11.8, 2.1, and 4.4 mg/L-N, respectively. Organic nitrogen was decreased by 62%, due to oxidation and synthesis. The average concentrations of ammonia in the influent, aerobic grab sample, and effluent were 11.3, 14.0, and 14.2 mg/L-N, respectively. The ammonia content increased by 25%, due to the ammonification process. Surprisingly, nitrate/ nitrite concentrations were all less than unity (Figure 14). These data show that very little if any nitrification of ammonia to nitrate occurred in The Tank.

Total Nitrogen was calculated by summing all forms of nitrogen. Figure 15 represents the results of TN concentrations as a function of tank section. The total nitrogen concentration for influent, aerobic grab sample, and effluent were 23.1, 16.1, and 18.6 mg/L-N, respectively. The analysis shows that the total nitrogen content was decreased by approximately 19%. This quantity of removal

is expected during normal metabolism (uptake of nitrogen for new cell material) and further indicates that virtually no nitrogen removal due to denitrification occurred.

Tables 16, 17, and 18 summarize the results of TKN, Ammonia-Nitrogen, and Nitrite/Nitrate values.

Table 16: Summary of TKN values (mg/L-N)

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	14.6	34.4	23.1	4.4
Aerobic Chamber	14.0	19.3	16.9	1.5
Effluent	14.0	24.6	18.6	2.9

Table 17: Summary of Ammonia-Nitrogen values (mg/L-N)

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	6.9	15.0	11.3	2.3
Aerobic Chamber	9.8	22.7	14.8	3.7
Effluent	9.0	18.0	14.2	3.4

Table 18: Summary of Nitrite/Nitrate values (mg/L-N)

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	0	0.16	0.03	0.04
Aerobic Chamber	0	1.17	0.04	0.06
Effluent	0	0.13	0.03	0.03

The negligible degree of nitrification can be related to inadequate retention time, low nitrifier fraction, and/or low DO. Required retention time for ammonia oxidation can be calculated using the following formula (Metcalf and Eddy, 1991):

$$\theta_N = \frac{No - N}{UXf_n}$$

where θ_N = retention time required for nitrification, time
U = substrate utilization rate, time^{-1}
X = concentrations of microorganisms, mg/L
No = influent TKN value, mg/L
N = the effluent TKN value, mg/L
f_n = nitrifier fraction

The above relationship indicates that retention time is directly related to the difference of TKN concentrations and inversely related to concentrations of microorganisms and substrate utilization rate. This equation was adapted for attached growth process by estimating the amount of microorganisms attached to the media (X). The substrate utilization rate will remain fairly constant once the microorganisms adjust to the given biological conditions specified by the waste.

The volume of microorganisms was estimated by multiplying the surface area of media filter by an estimated microorganism thickness between 0.05 mm to 0.1 mm. A range between 2480 mg/L and 4950 mg/L microorganisms were calculated by multiplying the volume of microorganisms by an assumed density of 1 Kg/L (8.34 lb/gal) and then dividing by the volume of the aerobic chamber. Typical microorganism concentrations are between 3000 and 10,000 mg/L. Using the equation above, an HRT between 20 minutes and 1.3 hours was calculated. The average TKN concentration of 23 mg/L, a typical nitrogenous substrate utilization rate of 0.8/day, a nitrifier fraction of 0.3, and the minimum

and maximum values of estimated microorganism concentrations were used to calculate the acceptable HRT based on the estimated range of microorganism populations. The 16 hour HRT that the UCZ-5 provides at 400 GPD should be more than adequate for carbonaceous and nitrogenous oxidation.

The Total Kjeldahl Nitrogen ratio (BOD_5/TKN) is also indicator of the ability of nitrogen to be nitrified. Because the aerobic chamber is preceded by anaerobic chambers reducing BOD concentrations, the BOD/TKN ratio is fairly low. A BOD/TKN ratio of 0.6 was calculated using the average BOD and TKN values measured in this study. This correlates to a nitrifier fraction of 0.3. It has been found that when the BOD_5/TKN ratio is greater than 5, the process can be classified as a combined carbon oxidation system and nitrification process, and, when the ratio is less than 3, it can be classified as a separate-state nitrification process (USEPA, 1975). To achieve nitrification, all that is required is that maintenance of conditions suitable for the growth of nitrifying organisms (Metcalf and Eddy, 1991). According to the Total Kjeldahl Nitrogen ratio, the UCZ-5 contains a satisfactory amount of nitrifying bacteria in a combined oxidation environment.

If a sufficient number of nitrifying bacteria are present, the aerators in the treatment system must have additional capacity to satisfy the nitrogenous oxygen demand in addition to the carbonaceous oxygen demand for nitrification to occur (Foree, 1981). Adequate DO concentration to provide sufficient molecular oxidation is critical to conversion for nitrification. As discussed in

Section 5.3.1., the average DO concentration was approximately 3.4 mg/L with a standard deviation of 1.3 mg/L.

The molecular oxygen required for complete nitrification can be roughly estimated using the following equation (Metcalf and Eddy, 1991):

$$\text{lb } O_2/\text{d} = Q(kS_0 + 4.57\ \text{TKN}) \times 8.34$$

where
Q = flow rate, Mgal/d
k = conversion factor for BOD for low loadings on nitrification systems (range is from 1.1 - 1.25)
S_0 = influent BOD_5, (mg/L)
TKN = amount of TKN to be converted

Using the expression above with an average flow rate 400 GPD, a conversion factor of 1.15, a worst case BOD_5 concentration of 30 mg/L, and an average TKN value of 17 mg/L, an oxygen requirement of 0.17 Kg O_2/d (0.38 lb O_2/d) was calculated.

An oxygen transfer correction factor of 0.34 was estimated by assuming a desired operating oxygen concentration of 2 mg/L, oxygen saturation concentrations at the average temperature of 25.4 °C, and 0.95 and 0.5 for β, and α, respectively (Equation (6) in Appendix A). Using the correction factor of 34%, the above aeration efficiency was decreased to 0.002 Kg O_2/W•d (3.3 lb O_2/hp•d). The blower provided by the manufacture provides approximately 0.006 Kg O_2/W•d (9.7 lb O_2/hp•d). An 85 Watt (0.11 hp) blower was calculated by dividing the amount of molecular oxygen required for complete nitrification by

the aeration efficiency. The existing motor supplied with the UCZ-5 (61W) is undersized by 40%. A 100 Watt blower would provide adequate oxygen concentration to promote complete nitrification with a 17% safety factor.

In an experiment involving an anaerobic attached growth column using synthetic septic tank effluent, more than 50% of nitrogen was removed when contents in the anoxic chamber were mixed continuously with an average HRT of 2 days. It was also found that nitrification efficiencies were reduced significantly when the ratio of BOD_5 to ammonia-nitrogen was increased and little nitrification occurred beyond a depth of 1.2 m in the attached growth column (Katers and Zanoni, 1998). Nitrification efficiencies in the UCZ-5 could conceivably be raised by doubling the total capacity of the anaerobic chambers (increasing retention time), installing a mechanical mixing unit in both anaerobic tanks, and increasing the blower capacity in the aerobic chamber.

5.9 PHOSPHORUS

Analyses of orthophosphate (OP), otherwise known as reactive phosphate, and total phosphorus (TP) concentrations were performed on influent and effluent composite samples and aerobic chamber grab samples. Seven samples were collected at random intervals from February 5, 1999 through March 22, 1999. Figure 16 represents orthophosphorus content contained in the influent, aerobic section, and in the effluent. The diamonds and circles represent the maximum and minimum values attained for the samples analyzed,

respectively. The averages are marked by a dash. The average influent, grab, and effluent OP concentrations were 3.4, 2.4, and 2.5 mg/L-P with standard deviations of 0.5 mg/L-P for each. The average orthophosphate concentration was reduced by 26%.

Figure 17 shows total phosphorus present in the influent, aerobic grab sample, and effluent. The measurement for orthophosphate and total phosphorus were performed on the same day samples. The average TP concentrations for the influent, grab, and effluent were 4.6, 4.3, and 3.8 mg/L-P with standard deviations of 1.5 mg/L-P for each. The average total phosphorus reduction was 27%. Tables 19 and 20 summarize the results represented on Figures 16 and 17.

Table 19: Summary of Orthophosphorus (mg/L-P)

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	2.8	4.0	3.5	0.5
Aerobic Chamber	1.9	3.5	2.4	0.5
Effluent	2.0	3.4	2.5	0.5

Table 20: Summary of Total Phosphorus (mg/L-P)

Parameter	Lowest Value	Highest Value	Average Value	Standard Deviation
Influent	2.3	6.5	4.6	1.5
Aerobic Chamber	3.0	7.3	4.3	1.5
Effluent	2.4	6.7	3.8	1.5

Typical phosphorus content for untreated domestic wastewater is 8 mg/L for TP and 5 mg/L for OP. Typical phosphorus reductions using conventional secondary biological processes are between 10% to 30%; however, when an anaerobic zone is followed by an aerobic zone, the microorganism exhibit phosphorus uptake above normal levels (Metcalf and Eddy, 1991). As discussed in section 2.2.5. Phosphorus Removal, the tank essentially operates similar to an A/O process (Mainstream Phosphorus Removal system). Aerated wastewater is recycled to the anaerobic chamber #2 at a rate of 135 gallons per hour, providing a recycle time of 2 hours for the aerobic chamber. At 400 GPD the HRT for the aerobic tank is 16 hours. Therefore, wastewater is cycled through the aerobic and anaerobic chambers a total of 8 times before being replaced. Exposure to alternating aerobic and anaerobic conditions stresses the microorganisms so that their uptake of phosphorus is above normal levels. When BOD to phosphorus (P) ratios exceed 10 to 1, reductions below 1 mg/L of phosphorus can be achieved. The BOD/P ratio for the UCZ-5 under average test conditions was approximately 4 to 1, providing above average phosphorus reductions.

Although reasonable reductions in phosphorus was experienced using the UCZ-5, land treatment is probably the most feasible means for complete removal of phosphorus from domestic wastewater (Foree, 1981).

5.10 OTHER TESTS

Oil and Grease can be problematic for biological systems; especially systems that contain filter media to promote attached growth processes. Table 21 below shows the results of the Oil and Grease analysis performed on three different days.

Table 21: Oil and Grease Results

	Influent				Effluent				
Date	Final wt. (g)	Initial wt. (g)	Volume (ml)	Conc. (mg/L)	Final wt. (g)	Initial wt. (g)	Volume (ml)	Conc. (mg/L)	Percent Reduction
12/18/98	147.871	145.826	370	5.5	91.070	90.736	375	0.9	83.9
12/28/98	147.503	145.807	252	6.7	104.015	103.755	270	1.0	85.7
12/31/98	148.456	145.816	300	8.8	104.320	103.763	254	2.2	75.1
Ave.				7.0				1.3	81.6
STDEV				1.7				0.7	5.7

The oil and grease concentrations were reduced by about 82%. The anaerobic filter media and anaerobic treatment is suspected to be the cause of such reduction.

NSF 40 also requires that the effluent be diluted 1:1000 with distilled water and be tested for color, odor, oily film, and foam. The effluent was tested for odor, oily film and foam on 28 October 1998, 7 December 1998, and 27 January 1999. NSF-40's criteria is that color shall not exceed 15 units, threshold odor is nonoffensive, oily film is nonvisible, and no foam shall exist during 6-month evaluation period. Color was not tested because the cost of platinum cobalt (potassium-chloroplatinate) required for Standard Methods 2120 B was prohibitive, but visual testing shows that color is clear. If fact, the color was

found to be clear visually without any dilution made to the effluent evident from the digital photo, Photograph 21. The results of the effluent from the UCZ-5 met the criteria for odor, oily film, and foam.

5.11 COMPARISON WITH CASE STUDIES

The following conclusions were derived from comparing Boyd's County Demonstration Project, USA (Case Study #1), Narity City, Japan (Case Study #2), and Sequencing Batch Reactor (SBR) Activated Sludge Process, Japan (Case Study #3) with results obtained in this study.

- The UCZ-5 exhibits superior TSS and BOD reductions of any units tested under Case Study #1.
- The UCZ-5 produces similar effluent BOD_5 and effluent concentrations as in the units researched under Case Study #2. Turbidity as in the Case Study #2 (transparency) was superior.
- As recommended in Case Study #1, installing and operating a WTP like the UCZ-5 requires a central management authority to provide oversight and compliance functions.
- Sludge production has not been eliminated, requiring outside sources to dispose.
- The UCZ-5's air diffusers and blower appeared to be more reliable than in Case Study #1.

- If economically and technically feasible, the SBR process presented in Case Study #3 shows the best SS, BOD, and nutrient reductions of any of the CSTR process, including the UCZ-5.

CHAPTER 6
COST ESTIMATES

6.1 INTRODUCTION

The cost estimate for the septic and WTP systems were based on an average flow rate of 400 GPD (required assumption for a 2 bedroom house in Hawai`i). A comparison between a septic tank with an attached absorption field (leach field) and a WTP with a deep absorption trench (seepage pit) has been prepared. A seepage pit following the WTP unit was selected instead of a leach field design because the superior effluent produced by the WTP allows disposal using a system with less treatment surface area, which produces a reduced foot print. Compact systems in the state of Hawai`i are attractive because of the high cost of real estate. Estimates were based on published estimates (Means, 1998) and typical contractor rates based on the island of Oahu (Nogato, 1999).

The septic system cost was based on installing a 1000 gallon pre-fabricated manufactured (fiberglass) tank. The footprint of the leach field following the septic tank is 20 ft wide by 30ft long. Using a trench sidewall capacity of 1.2 gal/ft•d and a 400 GPD average daily flow rate, 330 ft of disposal field trench length was calculated. Eleven, 30 ft. long trenches were separated approximately 2 ft apart within the leach field area. Each trench is 3 ft. deep and 2 ft. wide with 3 inch diameter Polyvinyl Chloride (PVC) perforated piping centered in the trench, 12 inches below ground. This allows a recommended hydraulic loading rate of 0.3 gal/ft^2•d (Metcalf and Eddy, 1991). The pipe

connecting the septic tank, house, and soil absorption field is also 3 inch diameter Polyvinyl Chloride (PVC). Approximately 30 feet of 3 inch PVC was estimated to make the connections between house, septic tank, and leach field. The trenches are filled with 24 inches of #3 gravel and the remaining depth is filled with 12 inches of burrowed soil material.

The cost estimate for the WTP was based on a 1,000 gallon pre-fabricated tank similar in operation to that of the UCZ-5. Effluent disposal was based on using a seepage pit. A total surface area of 235.5 ft^2 was estimated using an application rate of 1.7 $gal/ft^2 \bullet day$ for bottom and sidewall infiltration and the average daily flow rate of 400 GPD. Two standard sized 10' diameter by 5' tall pre-manufactured seepage pits with a distribution box splitting flow from the WTP to each seepage pit was used to satisfy the surface area requirement.

Estimates were based on normal soil conditions, no major construction obstacles, and delivery of materials and equipment within 12 miles from the contractor's storage yard.

Because the WTP is modular in design, the WTP system can replace a septic tank and use an existing absorption field. Also, an existing cesspool (essentially a seepage pit) could conceivably be cleaned out, modified, and reused by installing a WTP between it and the residence. If a consumer chose to or if the state regulated the use of a WTP using an existing soil absorption system, a consumer could save approximately $4,000.

6.1.1 SEPTIC/SOIL ABSORPTION FIELD ESTIMATE

Septic System Costs	Unit Cost ($)	Estimate ($)
Excavation (6 CY) (includes labor and equipment, delivery within 12 miles)	90/yd	550
Septic Tank (1000 gal plastic)	2,400 ea.	2,400
Dosing Pump (includes labor)	260 ea.	260
Misc. (3" PVC pipe, labor, materials, and geotextile)	----	1,000
		4,210
Absorption Field Costs		
Site survey (septic tank and disposal field)	----	500
Excavation (20' X 30' X 3' leach field)	30/yd	2,000
Gravel (50 CY)	24/yd	1,200
3" perforated piping (installation and labor)	4/ft	1,400
Misc. (Labor, materials, troubleshooting, etc.)	---	500
		5,600
Total System Cost		**$9,810**
Operation and Maintenance Costs	Unit Cost ($)	$/month
Sump Truck ($150 @ 2 times per year)	300/yr	25
Electricity (dosing pump)	24/yr	2
Total O&M Costs	**324/yr**	**27**

6.1.2 WTP/SEEPAGE PIT ESTIMATE

WTP System Costs	Unit Cost ($)	Estimate ($)
Excavation (6 CY) (includes labor and equipment delivery within 12 miles)	90/yd	550
WTP (1000 gal total) (includes aeration pump and delivery)	4,900 ea.	4,900
Sand (12 YD)	33/yd	400
Misc. (Tank and 3" PVC piping installation, troubleshooting, adjustments, etc.)	----	775
		6,625
Seepage Pit Costs		
Site survey	----	300
Excavation (35 CY)	54/yd	1,900
Seepage ring/cover	1,200 ea.	1,200
Gravel (12 CY)	24/yd	290
Sand (2 CY)	33/yd	70
Misc. (Installation labor and materials (distribution box), hookup, troubleshooting, etc.)		465
		4,225
Total System Cost		**10,850**

Operation and Maintenance Costs	Unit Cost ($)	$/month
Sump Truck ($150 @ 1time per year)	150/yr	13
Maintenance/Inspection (Service Contract)	150/yr	13
Electricity (aeration pump, 100W @ $0.12/KWH)	108/yr	9
Total O&M cost	**408/yr**	**35**

6.2 COST ANALYSIS AND DISCUSSION

The amount of excavation for the WTP installation was approximately the same as the 1000 gallon septic tank, about 6 cubic yards (CY). The cost estimate includes delivery of a wheel mounted hydraulically operated back hoe for excavation, labor to operate the equipment, and delivery. Unit cost for the excavation was estimated at $54/CY with the equipment owned by the contractor. The major cost difference in tank installation costs was the purchase of the WTP unit versus the septic tank - the WTP was $2,500 more. The cost listed under miscellaneous includes labor to install the 3 inch PVC piping between a residence, tank, and soil absorption system, any troubleshooting or adjustments in installation. Because installing a septic tank is a little more labor intensive, the cost was slightly more.

A soil analysis for the leach field/septic tank combination was believed to be more involved than a soil analysis for the WTP/seepage pit combination, so the estimate for the WTP was reduced by $200.

The amount of excavation for the leach field was approximately 10 CY more than the seepage pit. The difference is that the leach field is trenched while the seepage pit is completely excavated to provide volume for the two pre-manufactured seepage rings, which requires less labor. The cost for materials was about the same, except the cost associated for gravel. The trenches required a total of 50 CY of gravel where the seepage pit only required 12 CY - a $910 difference. The cost for gravel and the additional $200 estimated for site

inspection, was essentially the cost difference between the leach field and the seepage pit estimate. The overall installation cost for the septic/leach field system was $1,040 less than for the WTP system.

O&M costs were also estimated both systems. Recurring costs for both systems require a vacuum truck to dispose of sludge. A septic tank usually requires pumping twice a year where the WTP is anticipated to require cleaning once a year, if regular back washing and diffuser cleaning is accomplished. Also, the literature search shows that preventative maintenance and regular inspections (about every 3 months) are recommended for proper operation and effluent compliance of packaged systems (Kellam, Boardman, Hagedom, and Reneau, 1993). The O&M costs are 30% higher for the WTP unit.

In survey conducted on 54 aerobic packaged systems, it was found that proper, routine maintenance of household aerobic sewage treatment systems is essential for the proper operation of these units. Eighty four percent of the units tested produced poor quality effluent because of a defective aerator, diffuser, or timer. Most of these malfunctions occurred within the first four years of operations. Aeration units were divided into two categories. One category with blower and diffuser sealed within the aerobic chamber and the second category with the blower outside of the unit. Seventy seven percent of the unsatisfactory units were within the first category, while 23 percent were in the second category (Brewer, Lucas, and Prascak, 1978). Similar results were found in other studies (Kellam, Boardman, Hagedom, and Reneau, 1993). Fortunately, the pump

supplying air to the UCZ-5 is outside the chamber and pumps manufactured today are more reliable than in the past. However, a service contract was added to the estimate to assure compliance by a properly operating unit. The blower operated continuously during the 6 month testing period, only requiring the inlet filter to be dusted off about every 2 months. Little or no dust was discovered on the filter each time of inspection. Maintenance of this filter will prolong the life of the pump and will vary depending on the amount of dust present in the atmosphere. Routine maintenance items in the service contract should include inspecting the clarity, DO, and pH of the aerobic section and effluent (clarity only), inspecting condition of the anaerobic chambers (foam, clogging, etc.), any offensive odors emanating from the WTP, back flushing when required, inspection and cleaning the diffusers, dusting off the blower filter, and disposing of sludge at least once a year. Inspection, cleaning, and maintenance contracts by a certified contractor is mandatory in Japan.

Both estimates are similar in that a treatment tank followed by some sort of soil absorption process. The system installation costs were essentially the same - the WTP/seepage pit costing approximately $1040 more than the conventional septic tank/leach system. The main difference is that the WTP provides superior effluent to the soil absorption system (seepage pit) using a reduced footprint when compared to the septic tank/leach field system. This of course, comes at a slightly higher O&M estimate of $35/month - $8 more a month than the septic system.

CHAPTER 7
POTENTIAL MILITARY APPLICATIONS

7.1 MILITARY APLICATIONS

The military has changed roles in recent history for several reasons. The changing social-economic conditions brought upon by the completion of the Cold War has caused the military to decrease in size and has forced them to do more with less resources. The completion of the Cold War has also changed the diplomatic climate causing the military to become more involved in humanitarian assistance due to nations seeking sovereignty or due to natural disasters. Instead of a threat brought upon by a large military from an industrialized established nation, smaller less developed countries struggling to find independence are now being supported by a collective international organization; specifically the United Nations (UN). Under the North American Treaty Organization (NATO), many countries call upon the United States Armed Forces to provide humanitarian assistance or civic duties under the title "peace keeping forces". All services participate in this type of mission, but the role is best suited for the U.S. Army, U.S. Marine Corps, and small component within the U.S. Navy called Seabees. Many times under this type of role, large populations (refugees or victims of a natural disaster) are provided shelter within one or several small geographical areas.

Recent contingencies like Haiti, Bosnia, and Kosovo have used camps to provide infrastructure to as many as 40,000 people. These areas are commonly

called "Tent Cities" or "refugee camps". The biggest problem within these camps have not been overcrowding, but the transfer of diseases caused by unsanitary living conditions, including contaminated water sources caused by latrine discharge. The WTP unit could be used to improve the sanitary conditions of these campsites. "The Tank" could be connected to portable toilet facilities to provide better effluent conditions before disposal into a simple injection well or nearby surface discharge. Granted, this is not the best way to dispose of treated effluent, but would be a better method than disposing raw latrine sewage into a nearby ditch. Also, because the wastewater contains low organic contamination that was aerobically treated, the effluent is less apt to go "septic" preventing odors and annoying vector problems, like flies and mosquitoes.

With the continuing challenges of budget reductions, cost is an important factor in the implementation of such a system. Not only acquisition costs need to be considered, but also transportation and installation costs as well. In order for a system to be practical for this type of use, it needs to be transportable and capable of quick installation. The UCZ-5's integral unitized design makes it simple and compact helping it meet these criteria.

Such a system also needs to be cost effective. A study conducted by the U.S. Army Corps of Engineering Research Laboratory (USACERL) studying wastewater treatment using vault latrines, composting latrines, package plants, and Rotating Biological Contactors (RBC) concluded that conventional vaults should be retrofitted with aeration units, where electrical power was available.

The cost to retrofit these units was estimated at $2,000 per unit with $480/year energy costs (Smith and Scholze, 1984). Being the guest in another country in supplying them assistance, environmental considerations may outweigh an increase in cost to treat the wastewater being discharged. The WTP unit would not replace latrine type facilities, but would supplement them to improve discharged wastewater quality; thereby adding additional cost.

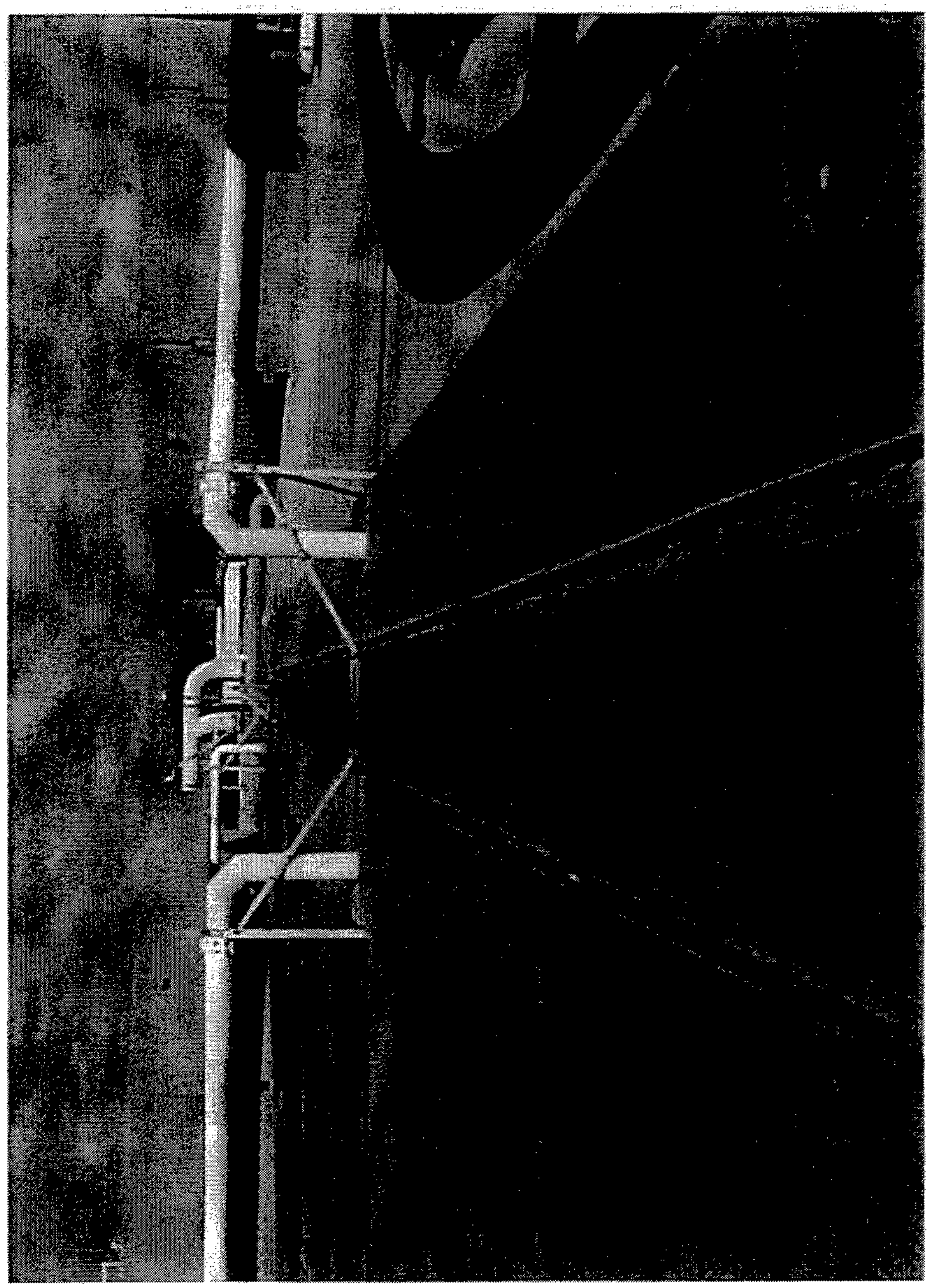
Before deciding if the WTP unit is feasible for the military in their role of providing humanitarian assistance, a detailed cost/benefit analysis will be required after all tests are complete on such a system. The results from the 6-month testing of the UCZ-5 seems promising in providing better sanitary conditions in operations that involve Tent Cities, camps, and remote recreational facilities located on military and governmental installations. The question is not "is it cheaper than latrine type facilities, "but" is it beneficial to treat wastewater (humanous solids, liquids, and gray water) to improve sanitary conditions of a camp, reduce solid handling requirements, eliminate offensive odors, and to lessen the contamination in soils and/or receiving waters." The answer to the second question appears to be a definite "yes".

CHAPTER 8
CONCLUSIONS

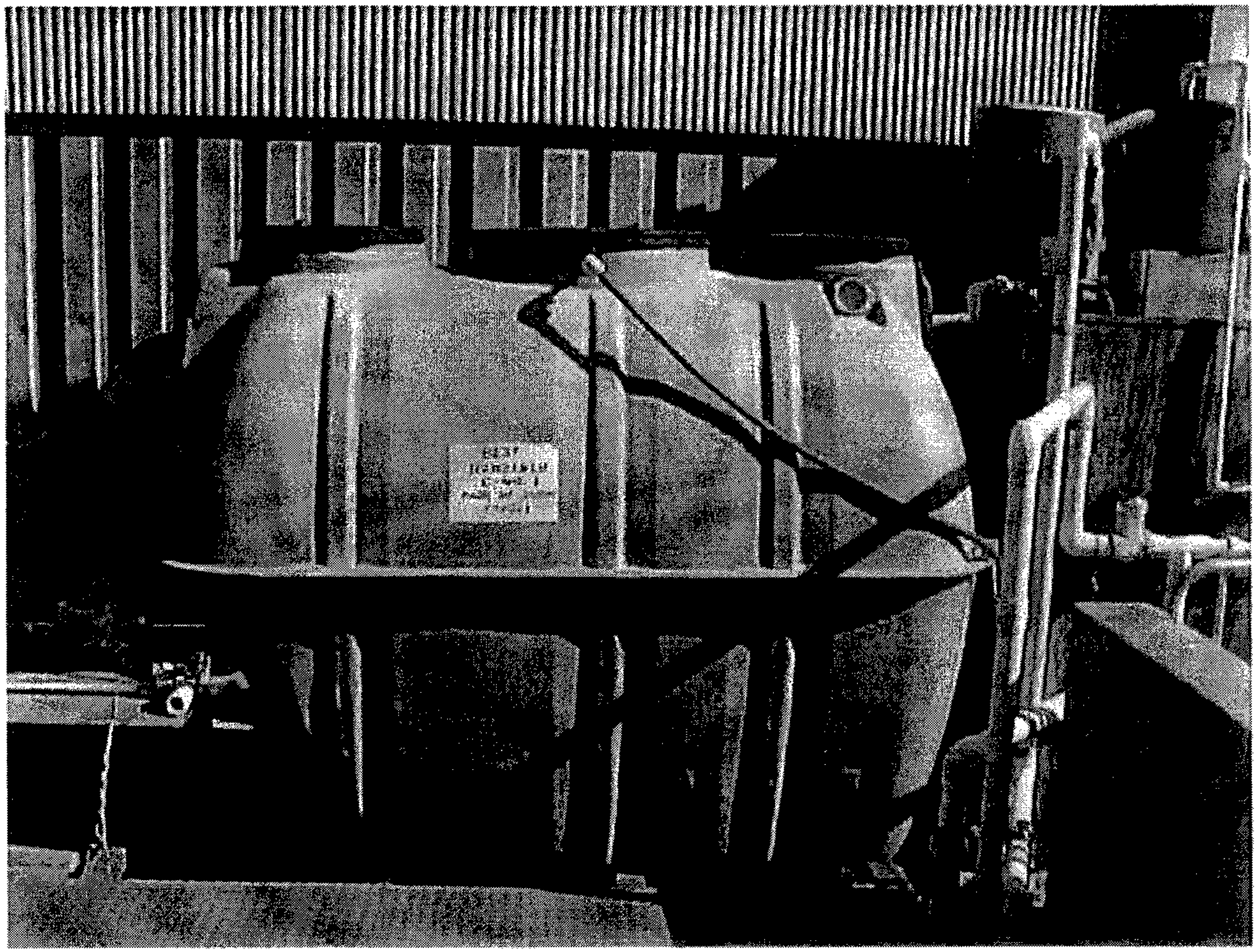
The UCZ-5 UTP unit performed exceptionally well during the 6-month testing period. The unit required little or no maintenance. The literature search and full-scale field study allow the following conclusions:

- The UCZ-5 unit reduces suspended solids, seattleable solids, BOD, turbidity, and orthophosphates to acceptable levels without producing offensive odors, oily film or foam under steady state conditions (400 GPD).
- Nitrogen and total phosphorus concentrations are reduced somewhat, but not completely eliminated.
- A soil absorption system after the UCZ-5 would be required if complete reduction of nutrients is desired.
- Regular maintenance and inspections of the UCZ-5 via a service contract is imperative to assuring acceptable effluent quality and to prevent mechanical failures.
- The feasibility of a WTP unit system is increased with multifamily housing units and partnering between homeowners and local, state, and federal agencies.
- Installation of mixing units in both anaerobic chambers, increasing DO concentration in the aerobic chamber, and increasing recycle rate are recommended for improved nitrification/denitrification.

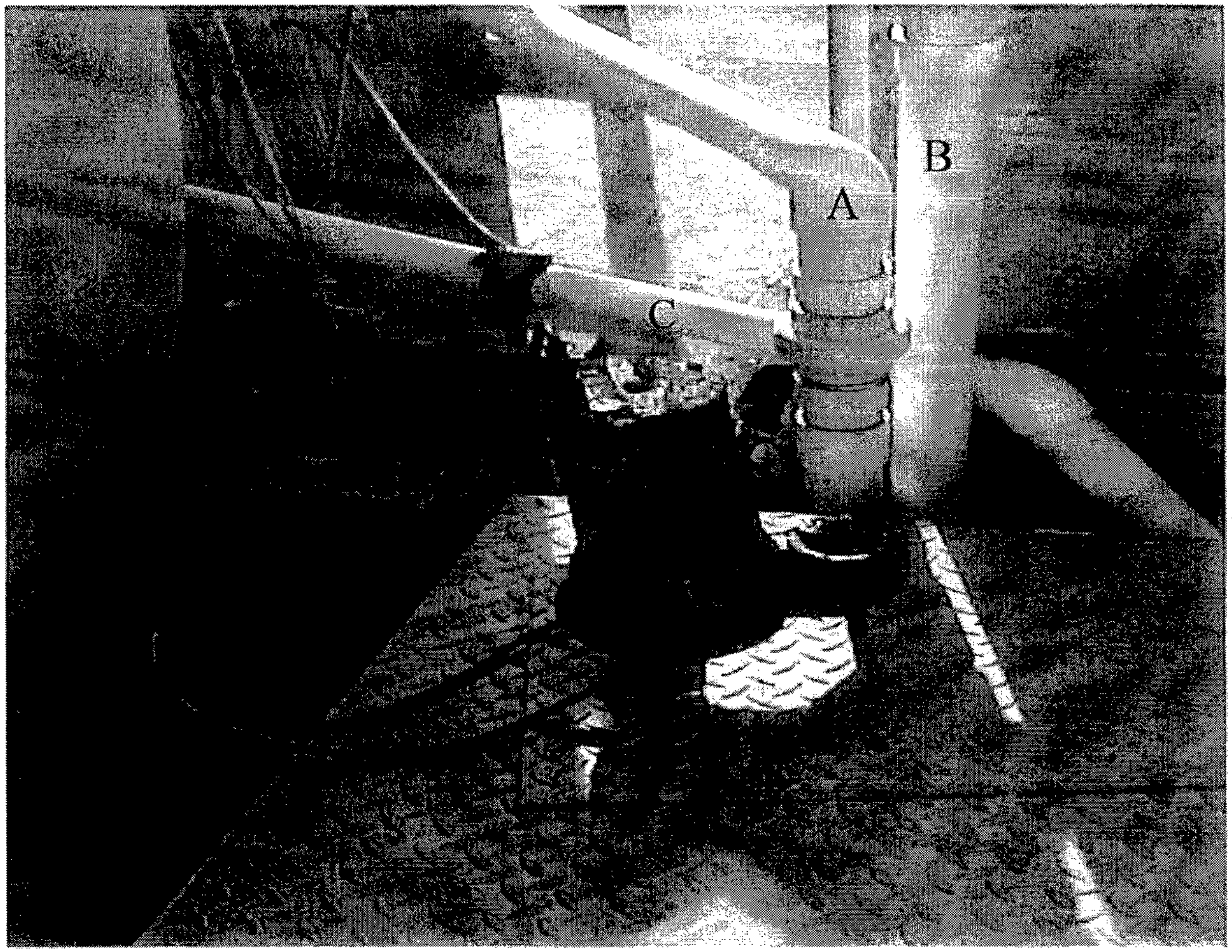
APPENDIX A: PHOTOGRAPHS



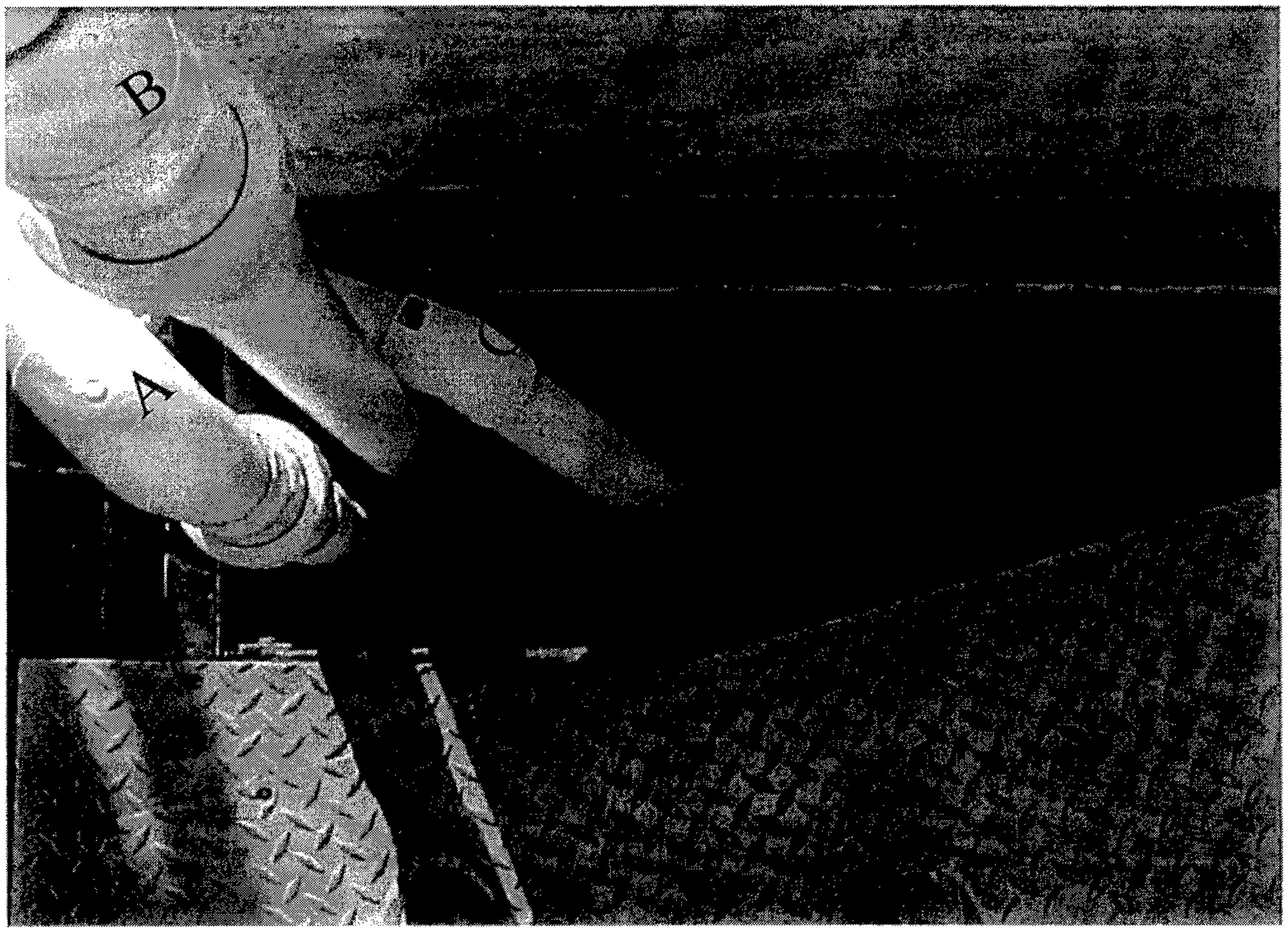
Photograph 1: Sand Island's Wastewater Treatment Plant influent channel.



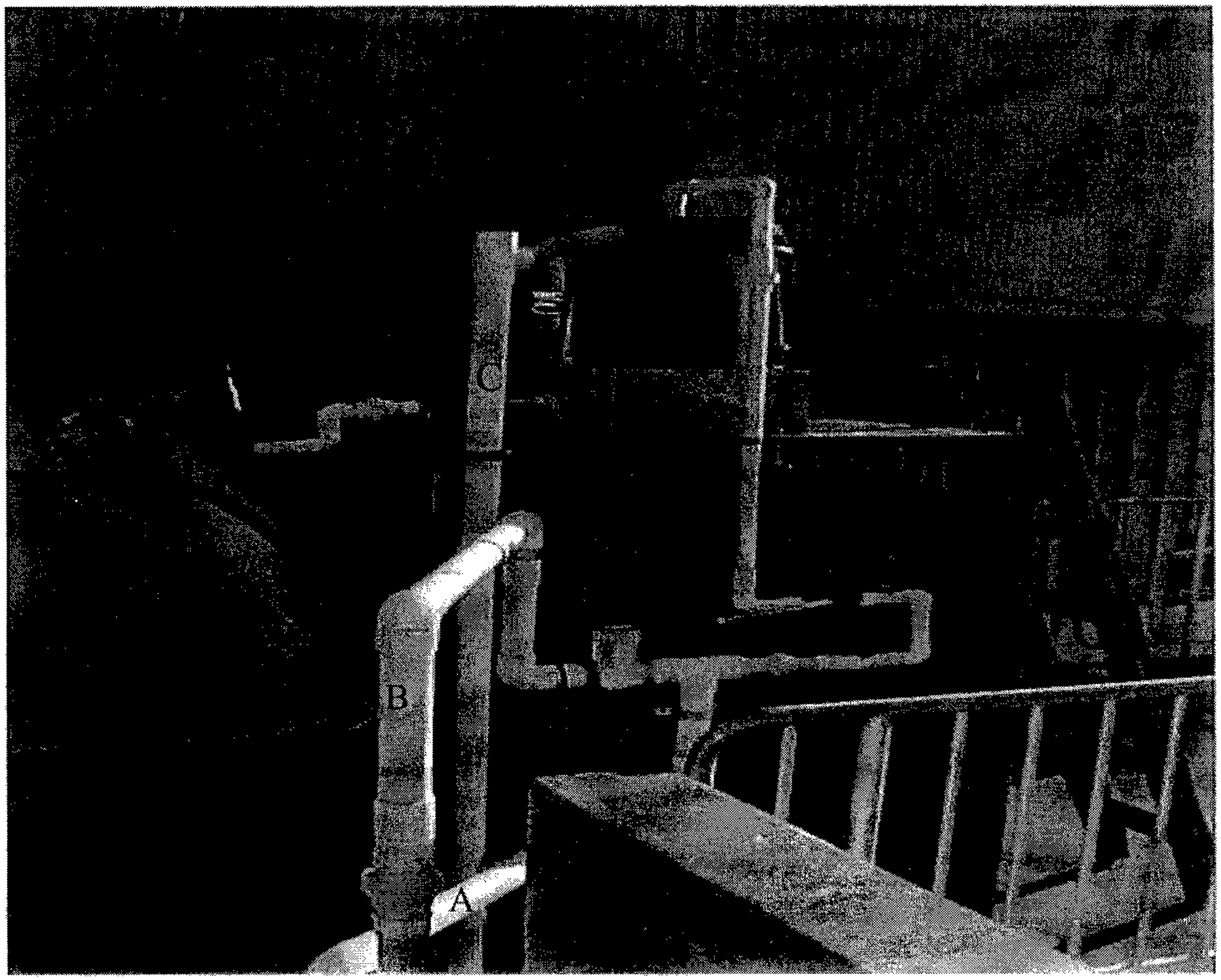
Photograph 2: BEST UCZ-5 Wastewater Treatment Package Unit



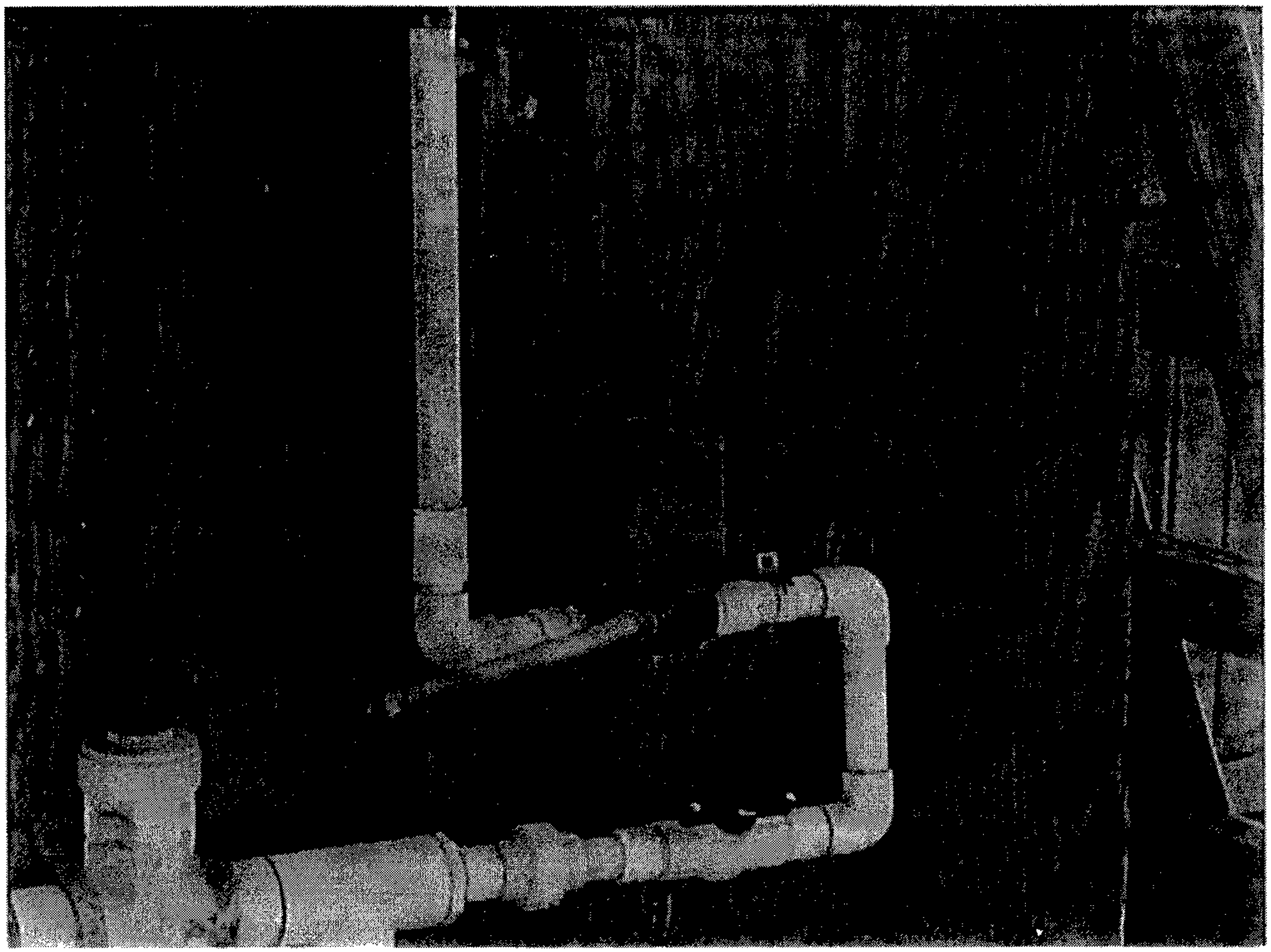
Photograph 3: Sump pump used to draw wastewater from SIWWTP influent channel.



Photograph 4: Influent access to Sand Island Wastewater Treatment Plant.

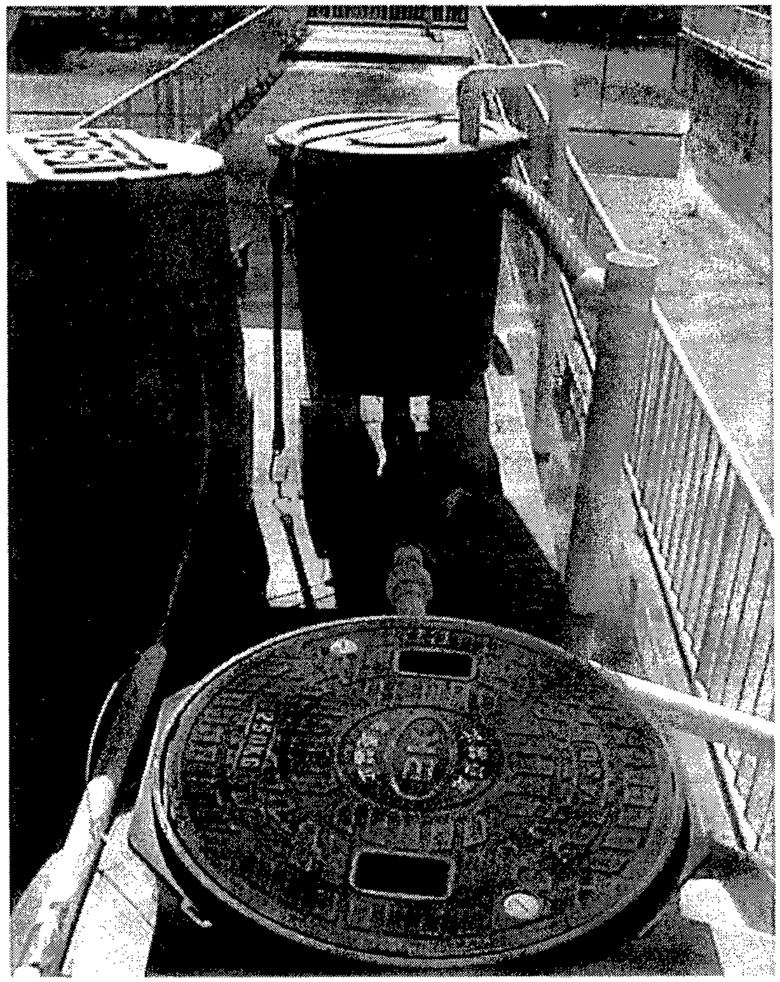


Photograph 5: Bucket apparatus setup and appurtenances.



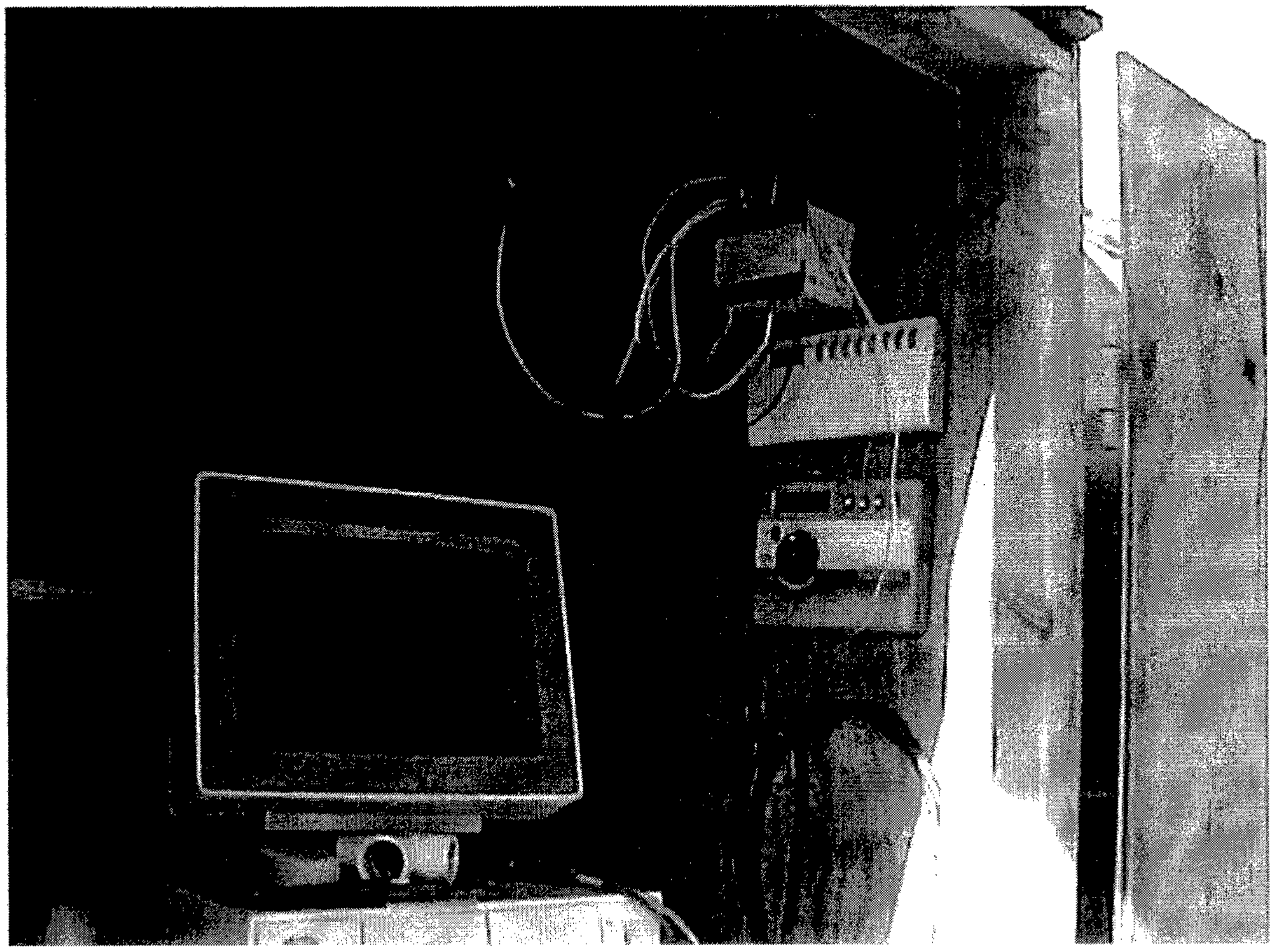
Photograph 6: Valve controlling flow from influent channel to bucket apparatus.

Photograph 7: Bucket apparatus to collect raw influent from SIWWTP.

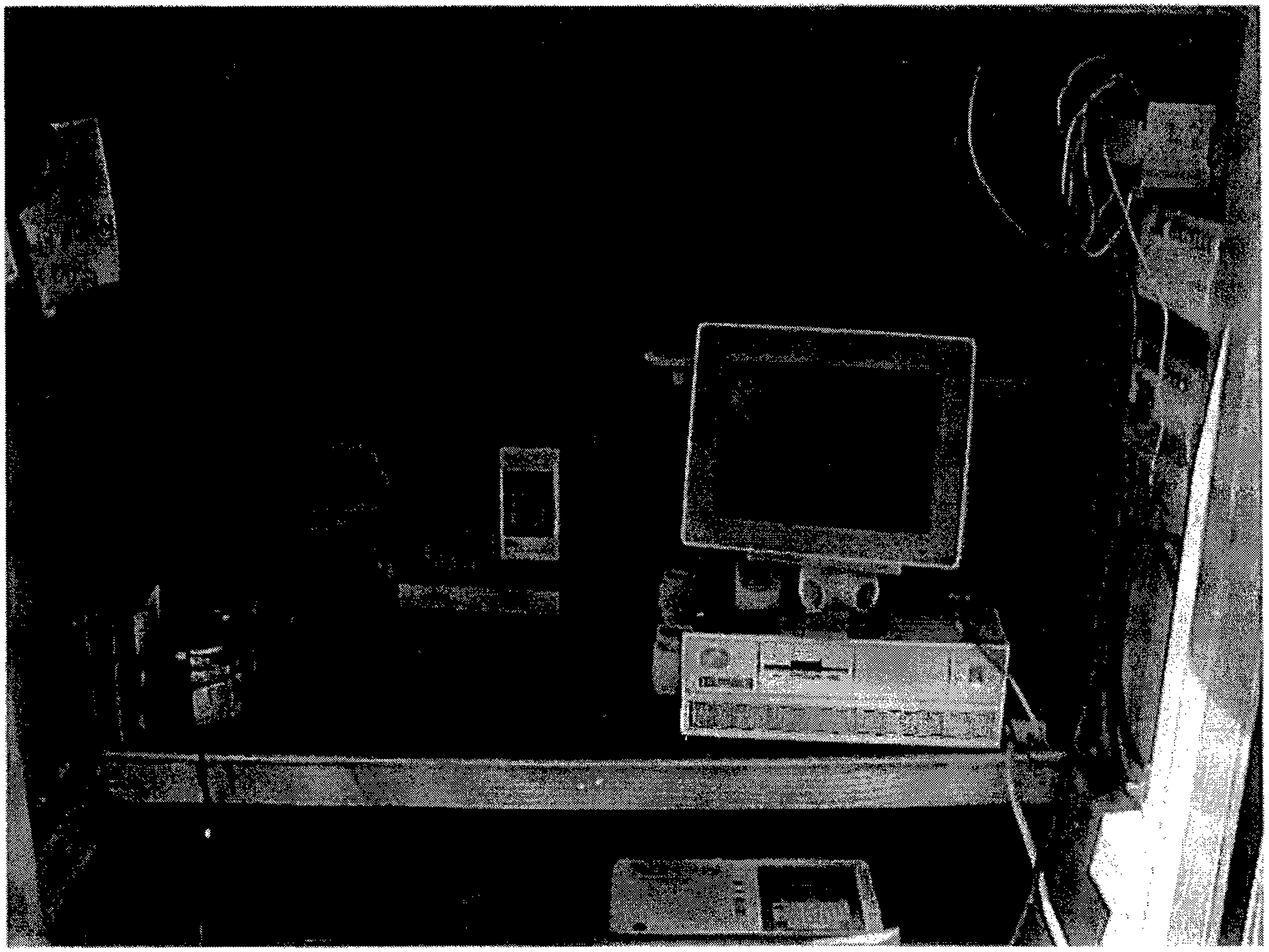


Photograph 8: Bucket apparatus, influent valve, effluent sampler, and tank inlet.

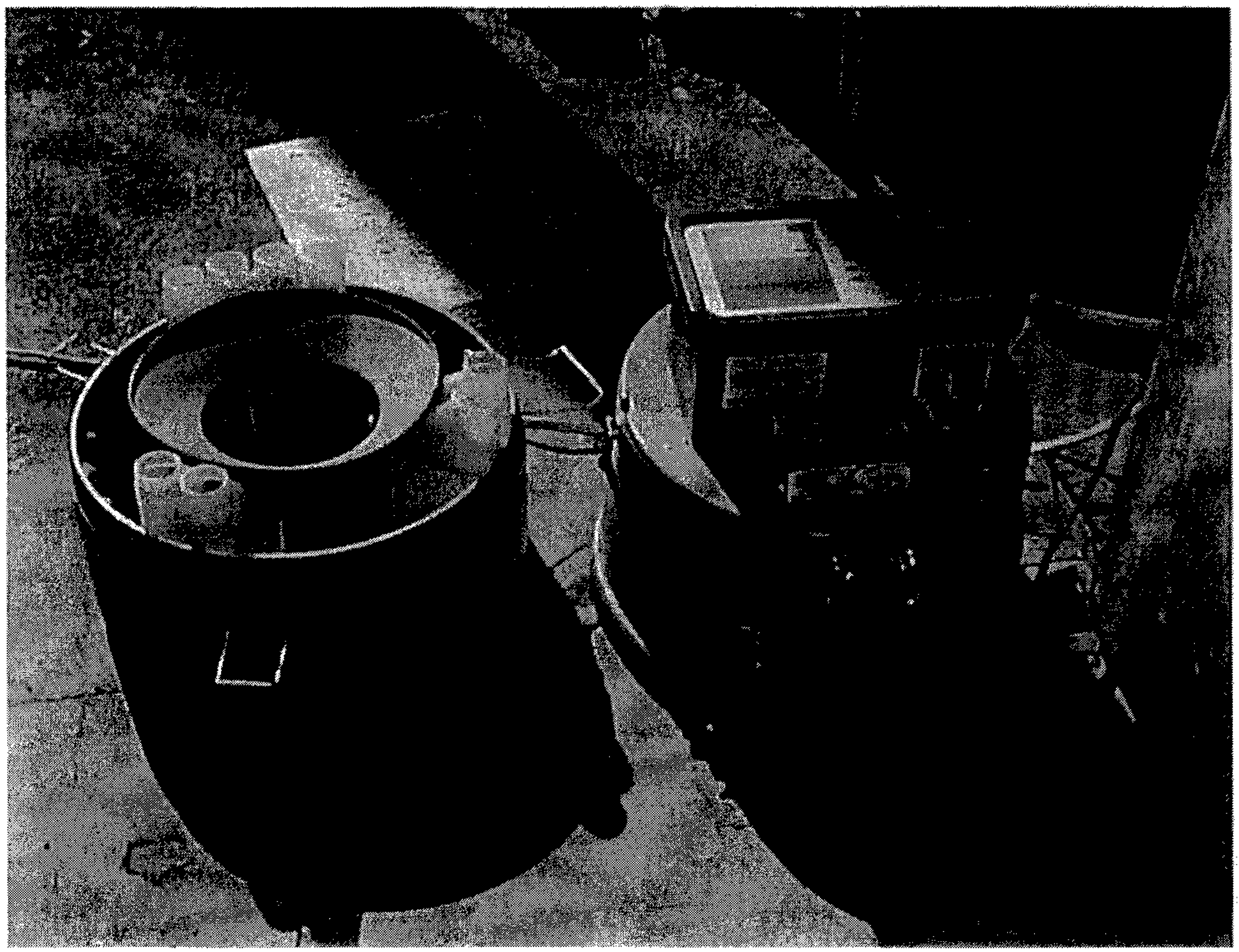
Photograph 9: Valve controlling flow from bucket apparatus to WTP with ISCO 3700 sampler to collect effluent in background.



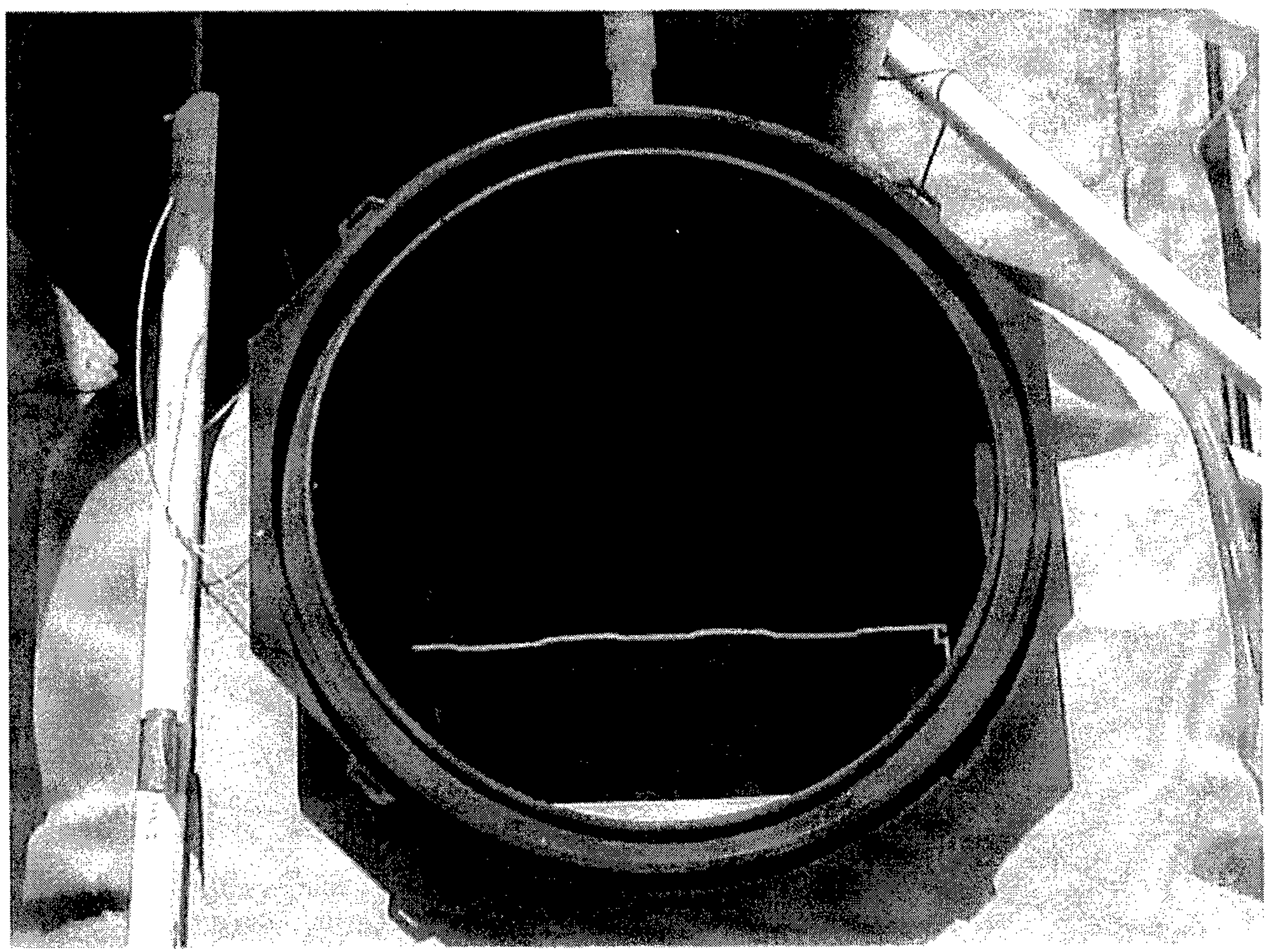
Photograph 10: Computer, rain timers, and PLC.



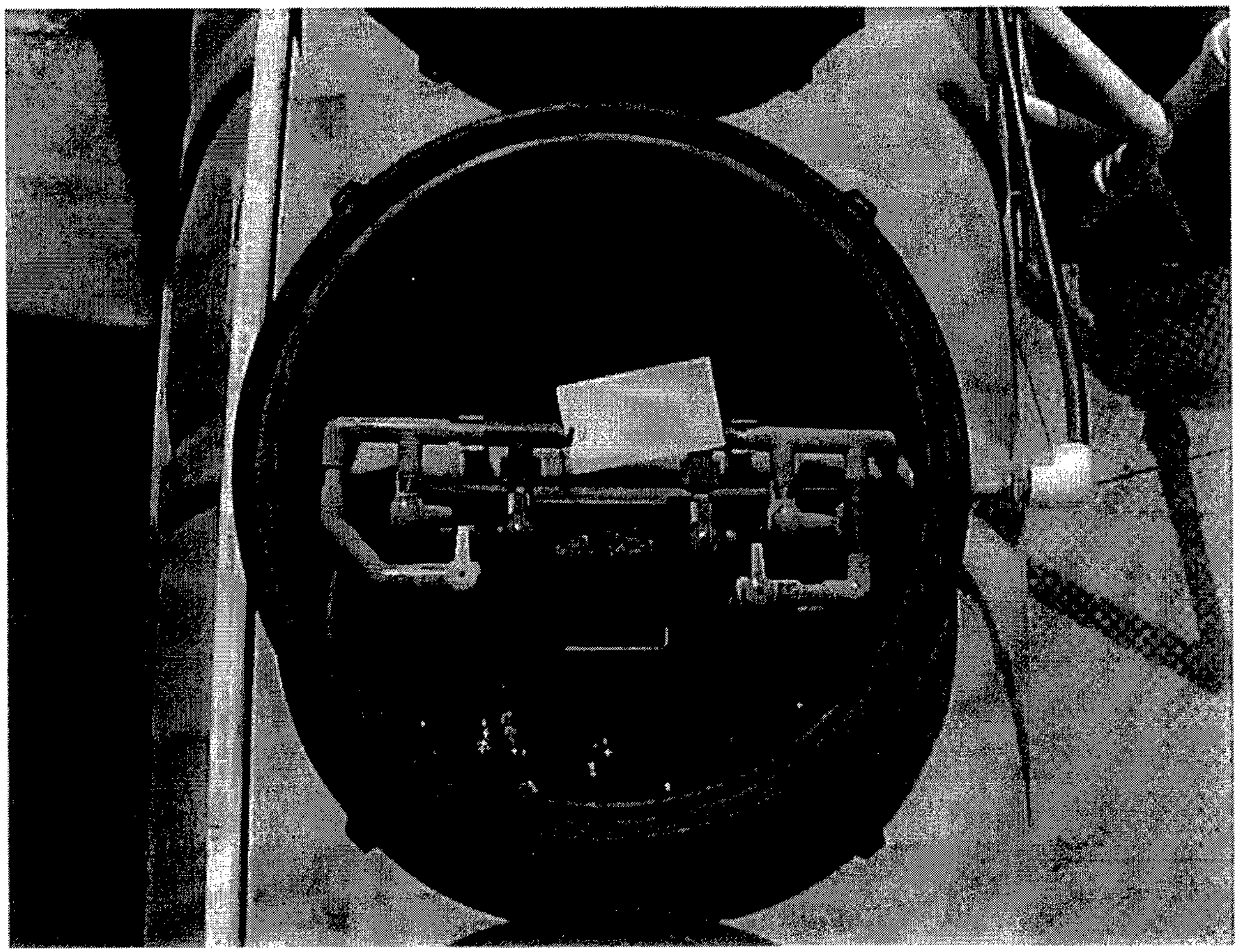
Photograph 11: Storage shed containing computer, sampler, and controlling devices.



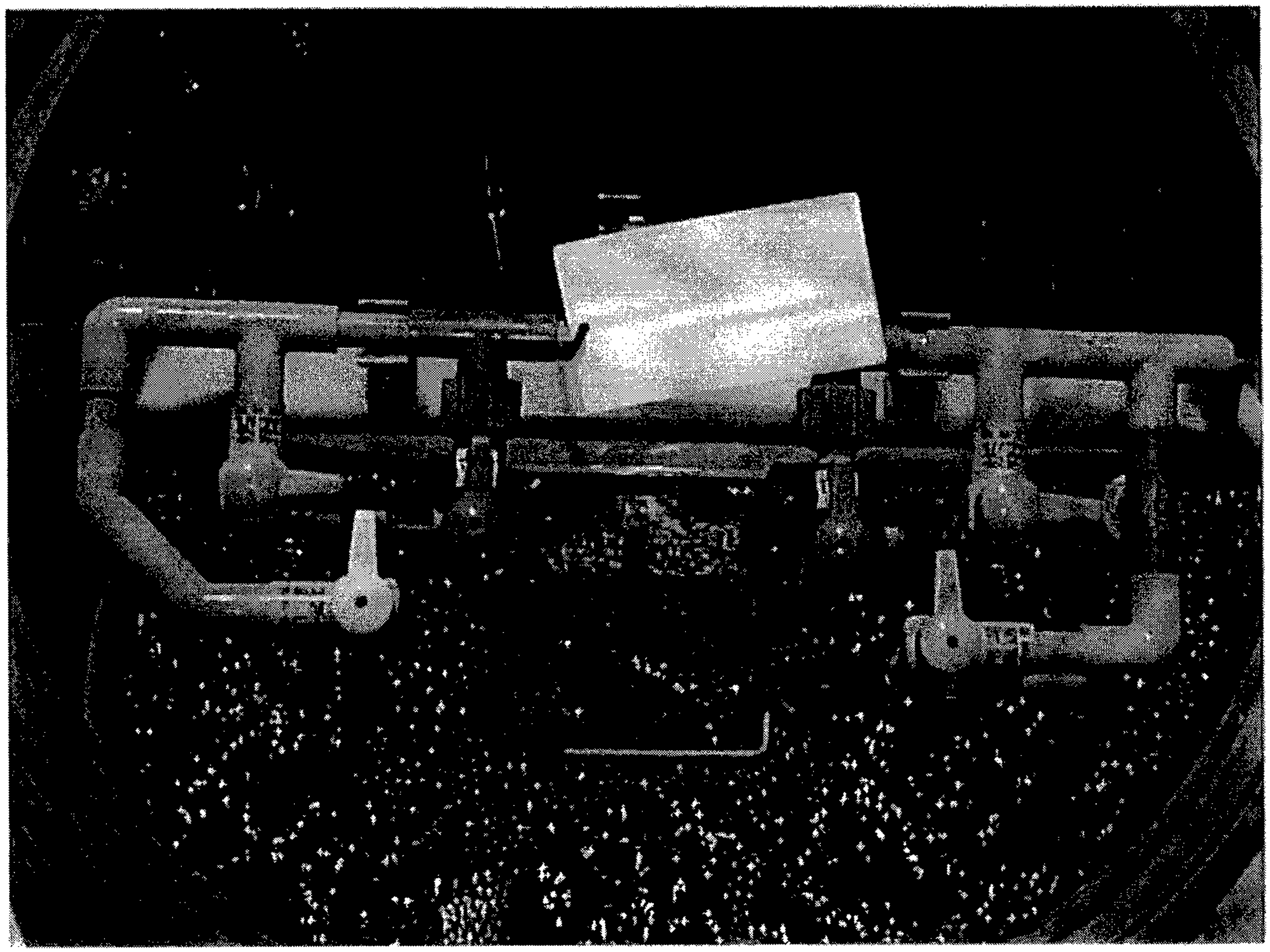
Photograph 12: ISCO sample used to collect wastewater (influent).



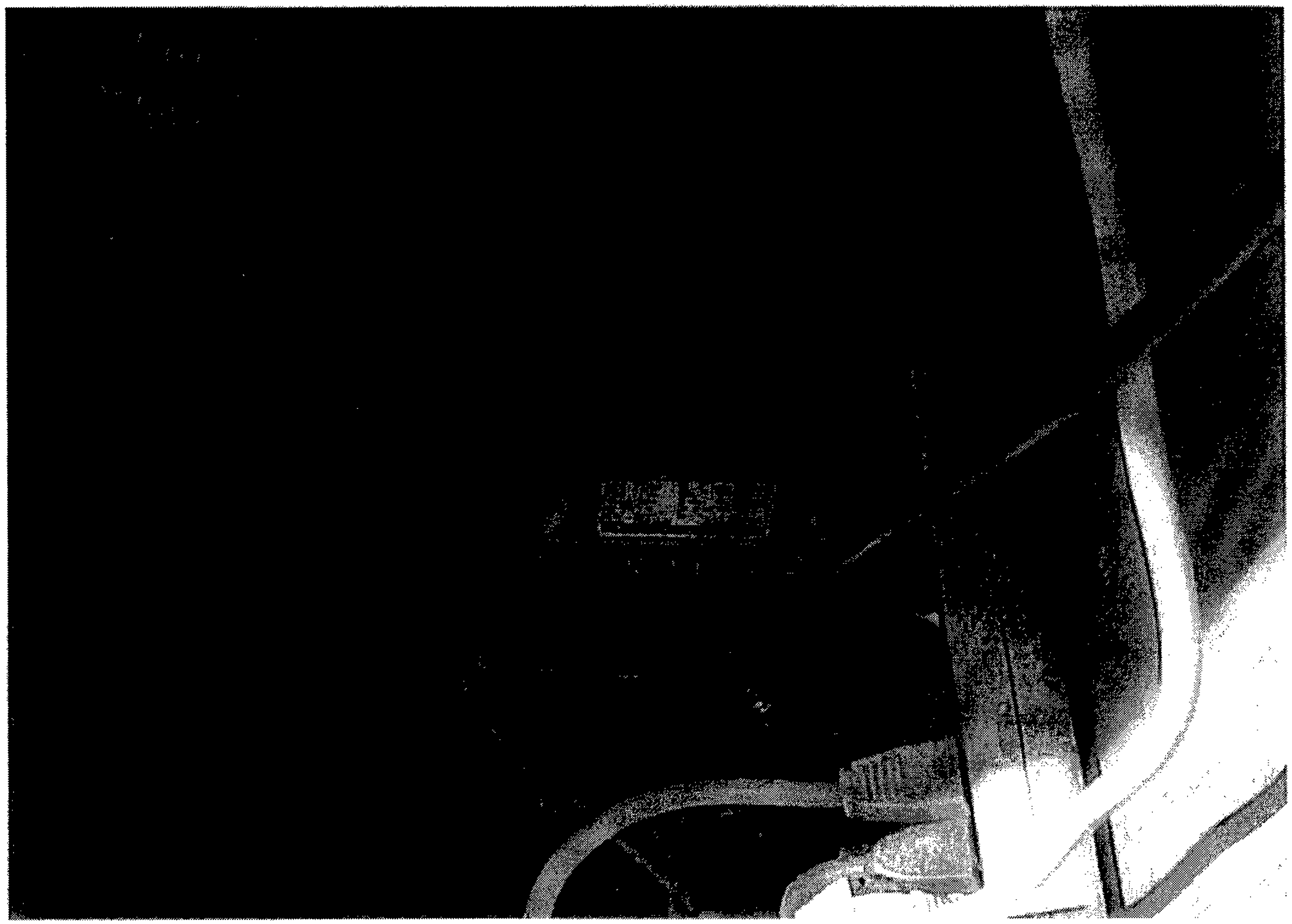
Photograph 13: Anaerobic chamber #1 with influent connection attached.



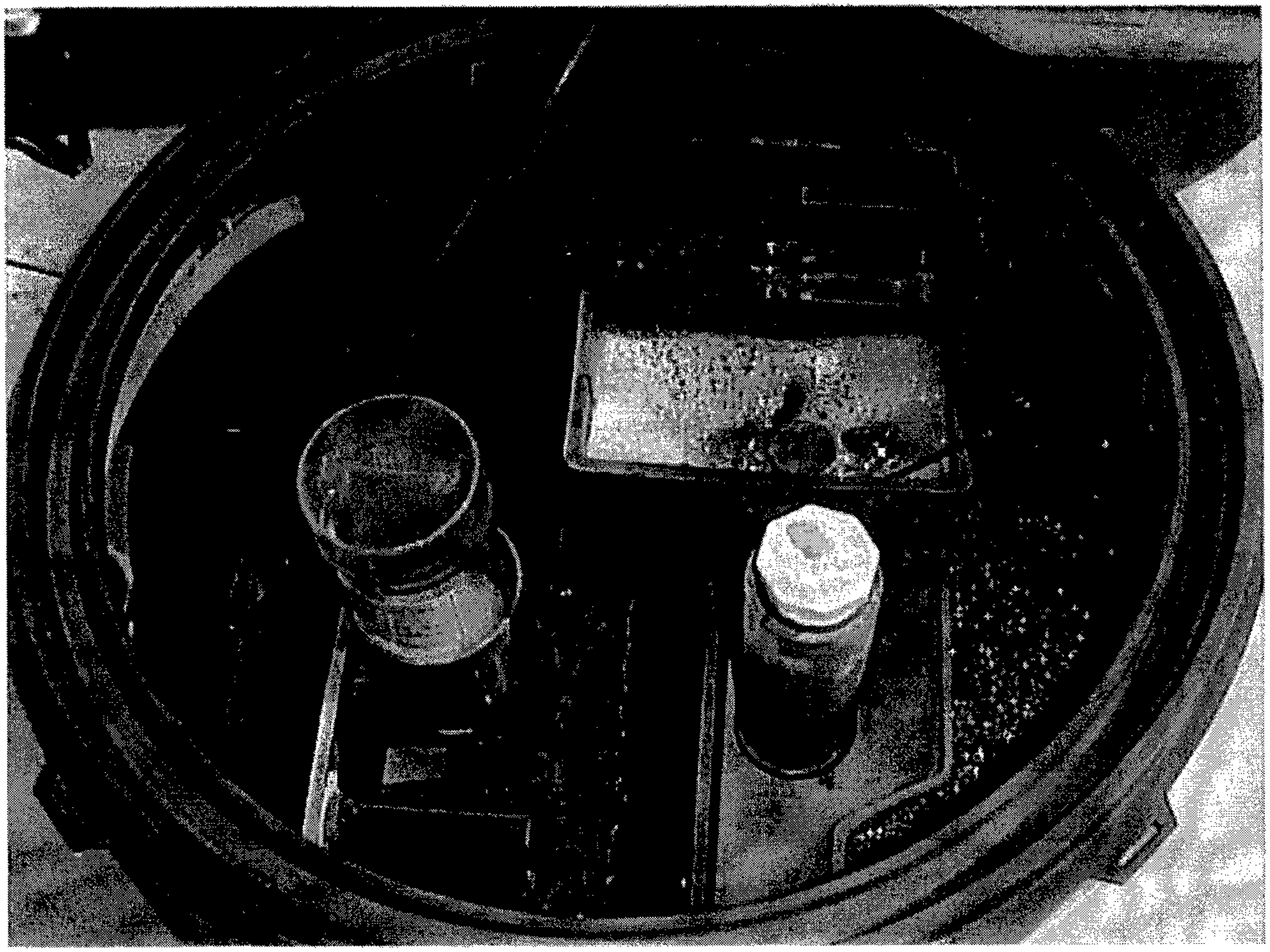
Photograph 14: Anaerobic chamber #2 and Aerobic chamber showing control valves.



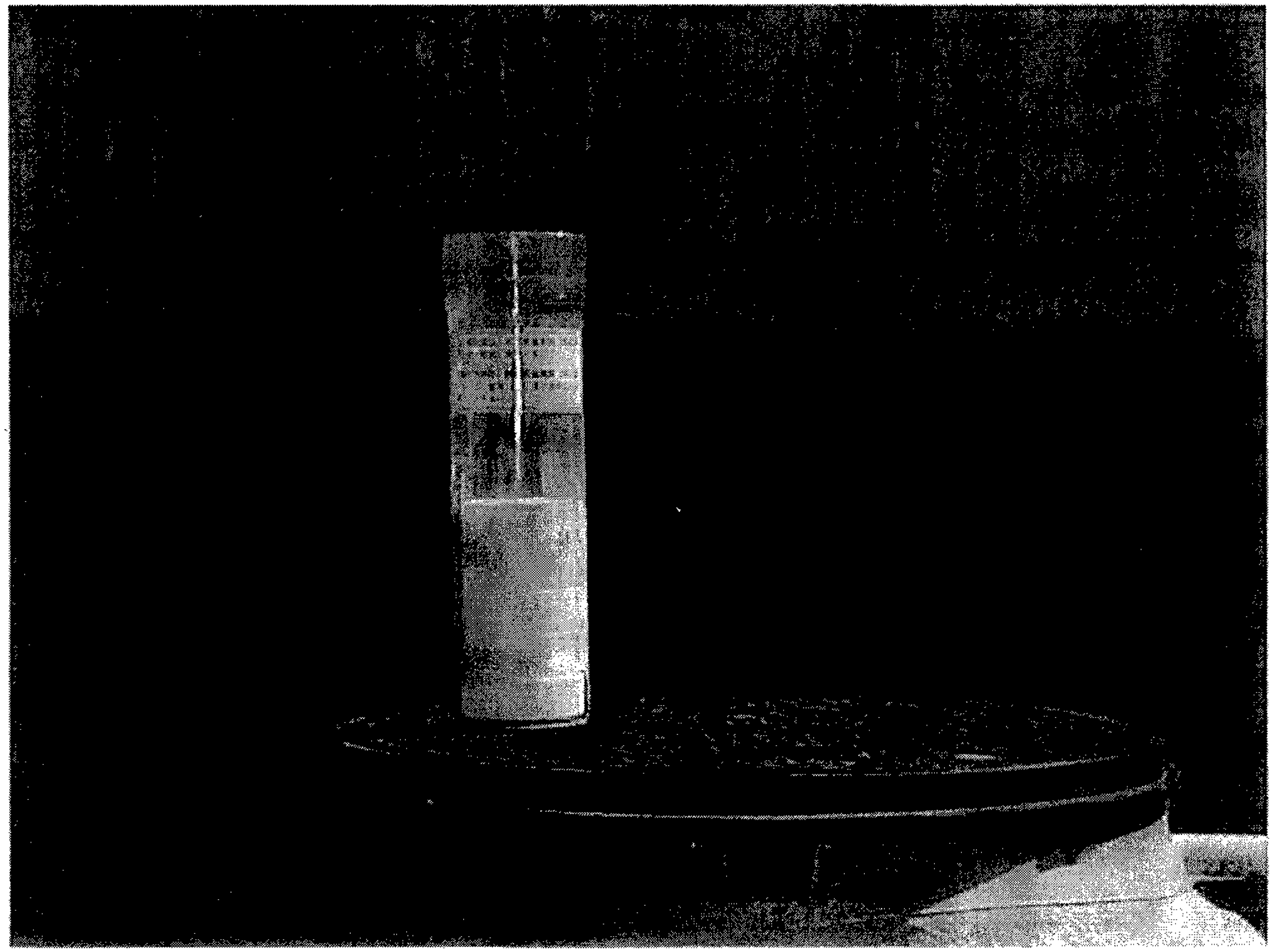
Photograph 15: Aerobic chamber showing control valves and aerobic treatment.



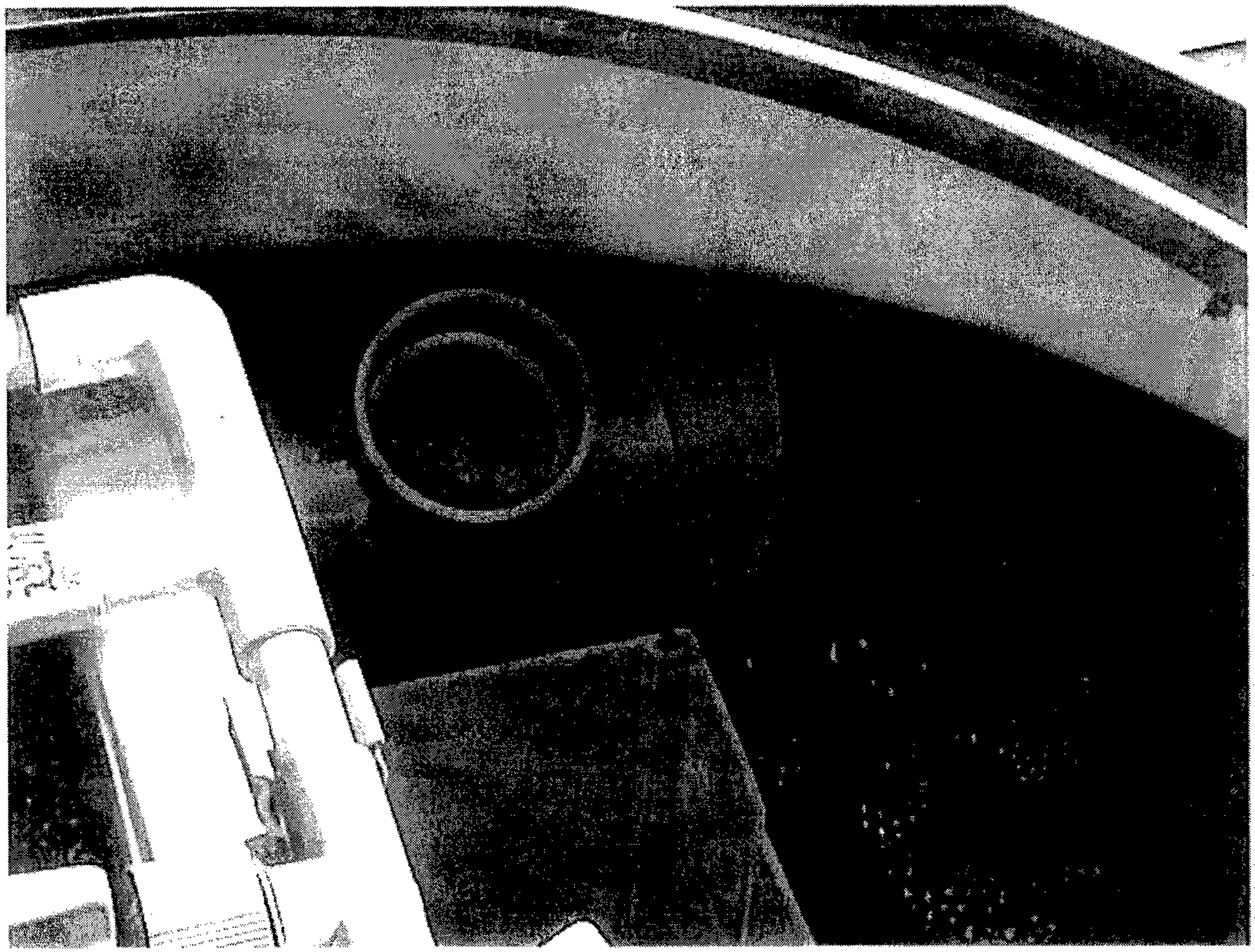
Photograph 16: Blower connected to aeration chamber via plastic tubing.



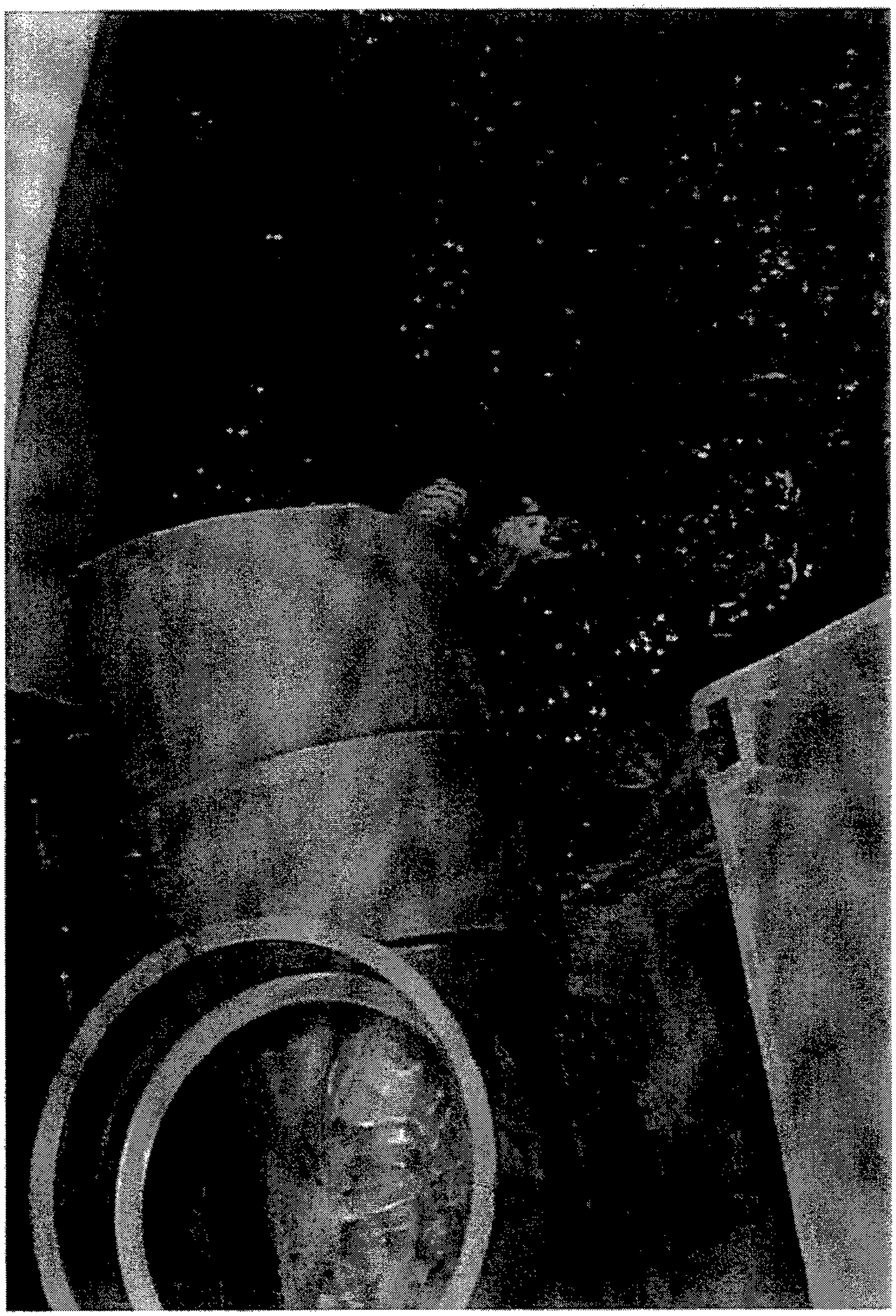
Photograph 17: Access to settling chamber, chlorine chamber, and recycle pumping container



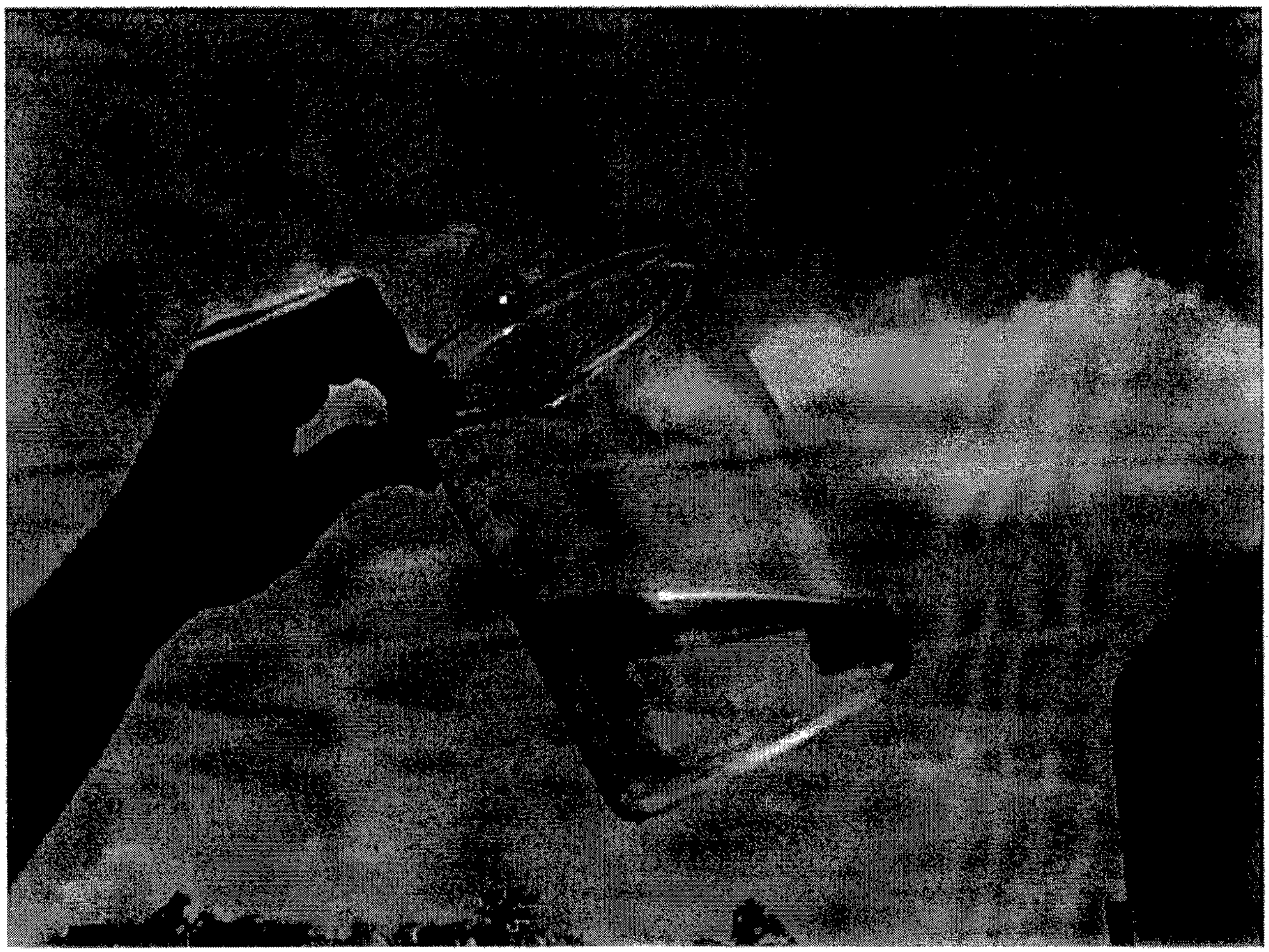
Photograph 18: Chlorine tablets and insert container displayed over access cover to chamber #3.



Photograph 19: Recycle line dispensing treated water to anaerobic chamber#2.



Photograph 20: Close-up of recycle line



Photograph 21: Treated effluent displayed in 1000-ml beaker.

APPENDIX B: FIGURES

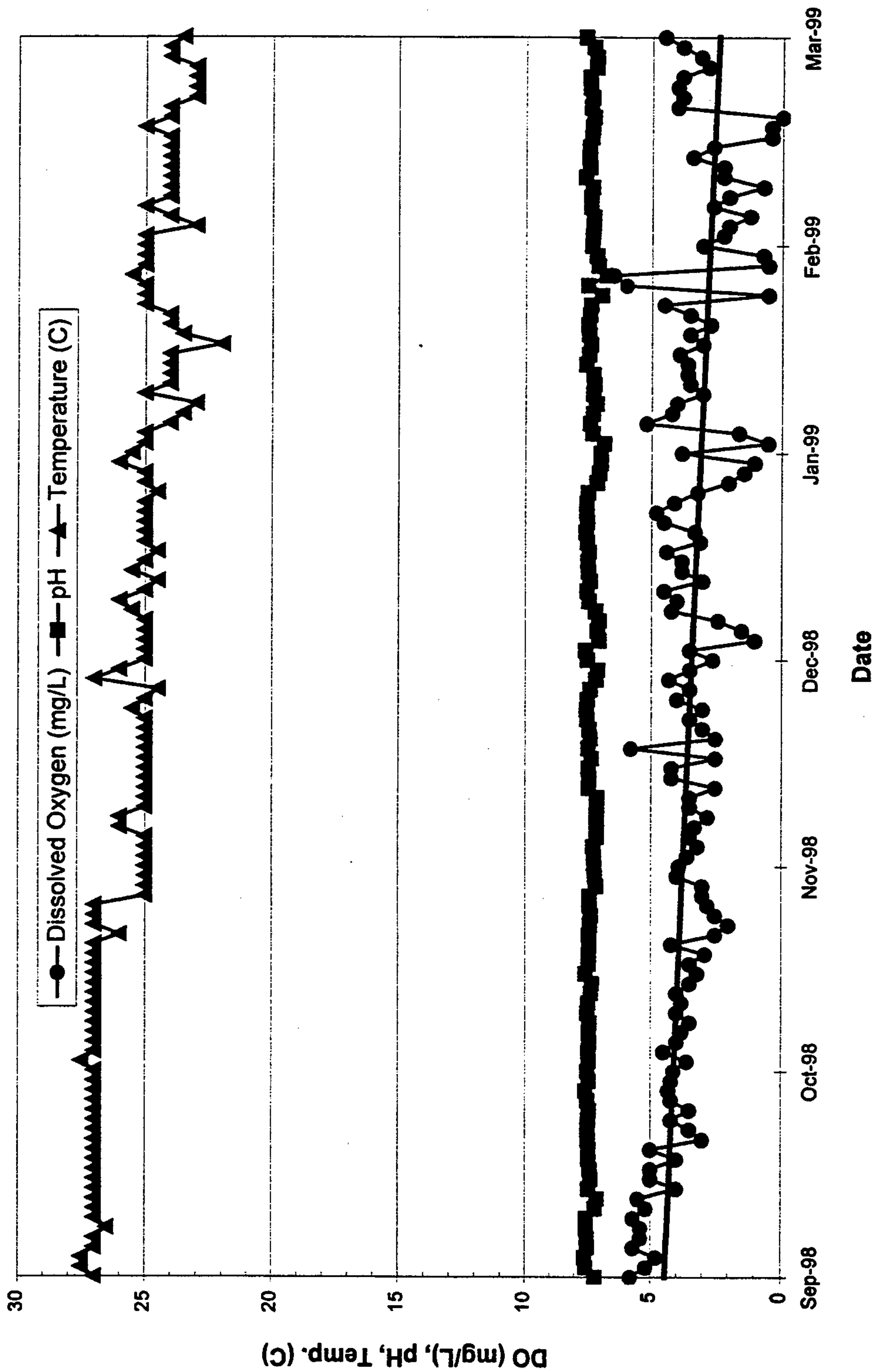


Figure 2: In-situ Measurements

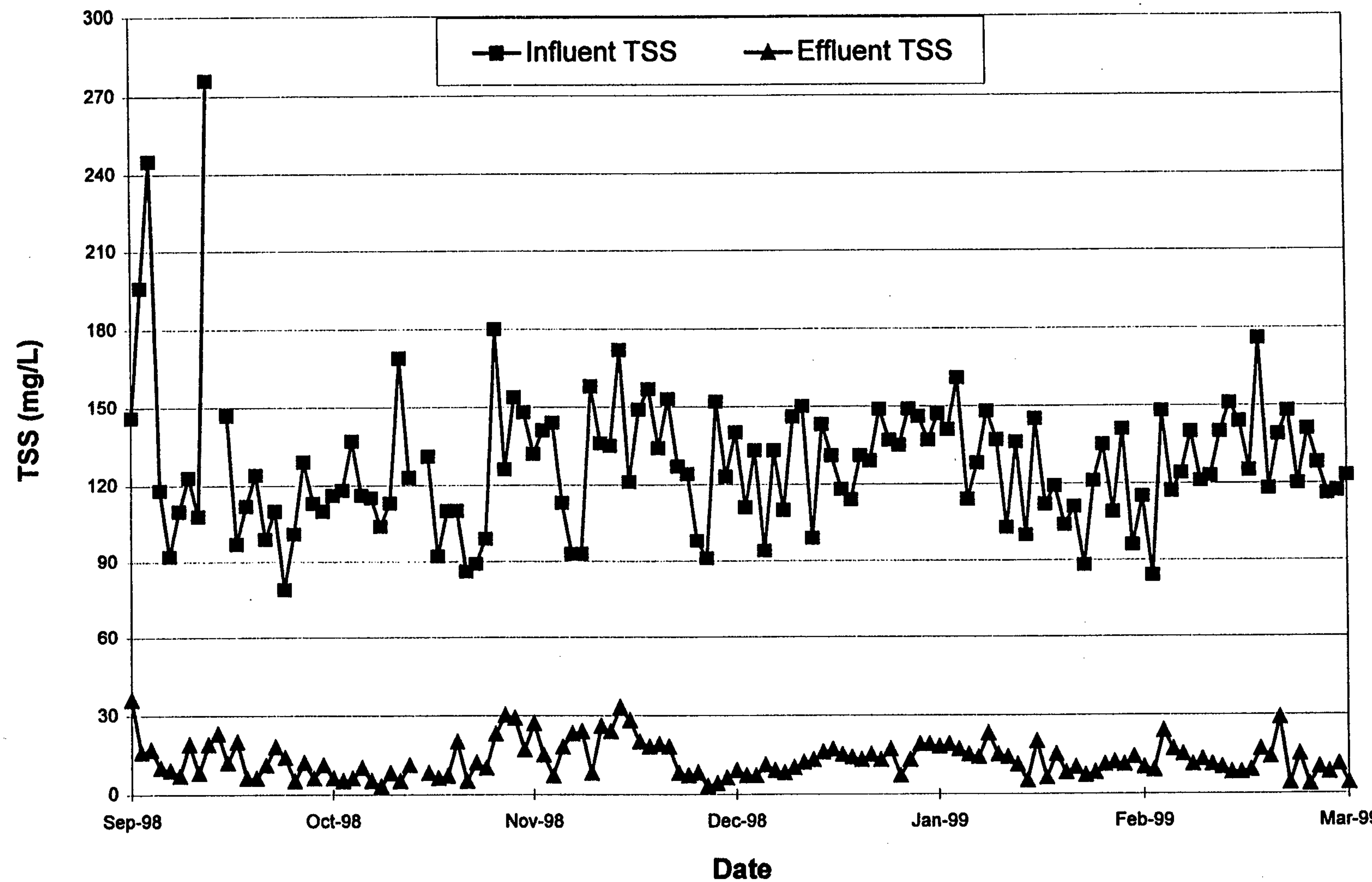


Figure 3: Total Suspended Solids (TSS) Concentration

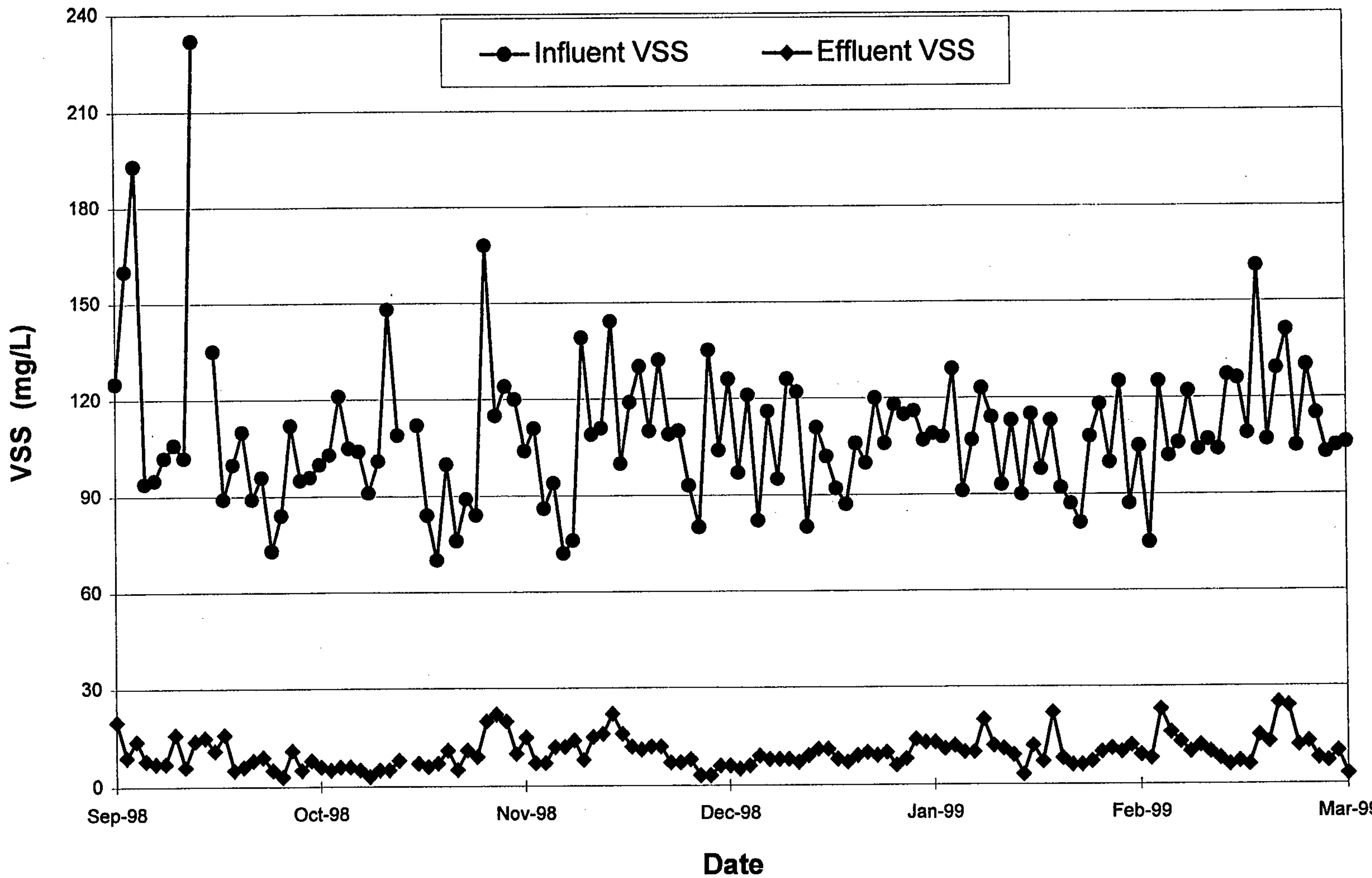


Figure 4: Volatile Suspended Solids (VSS) Concentration

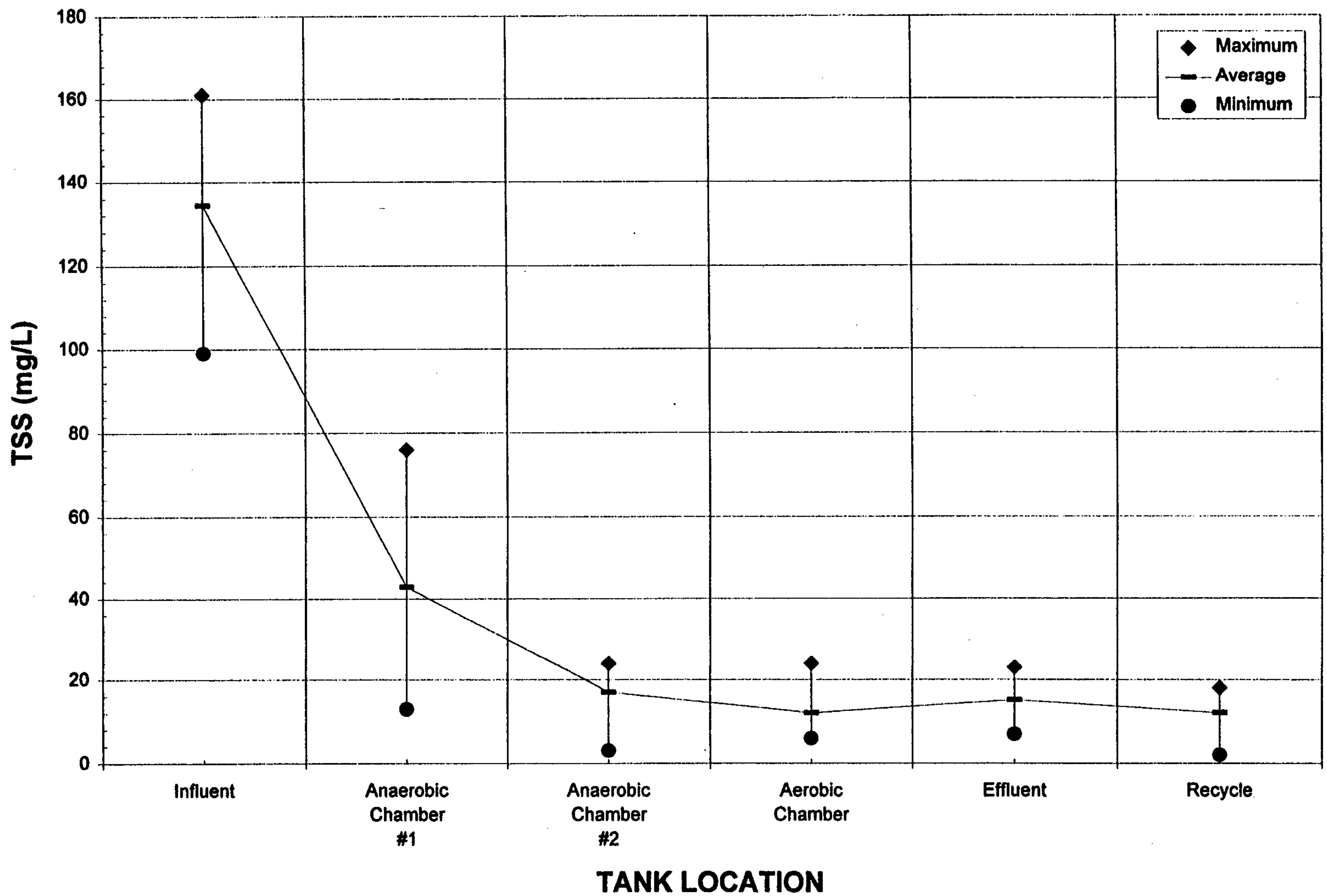


Figure 5: TSS Analysis by Chamber

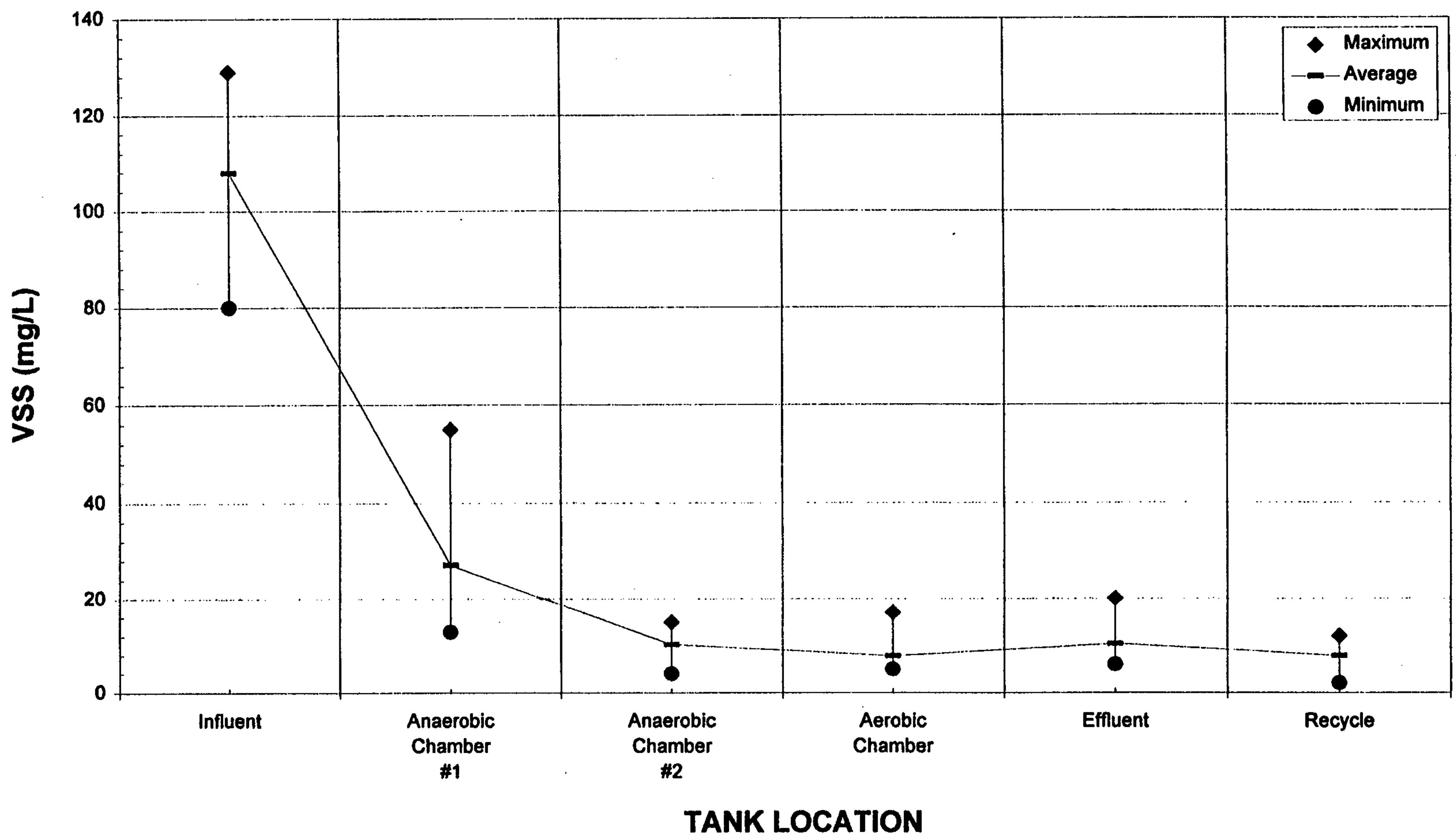


Figure 6: VSS Analysis by Chamber

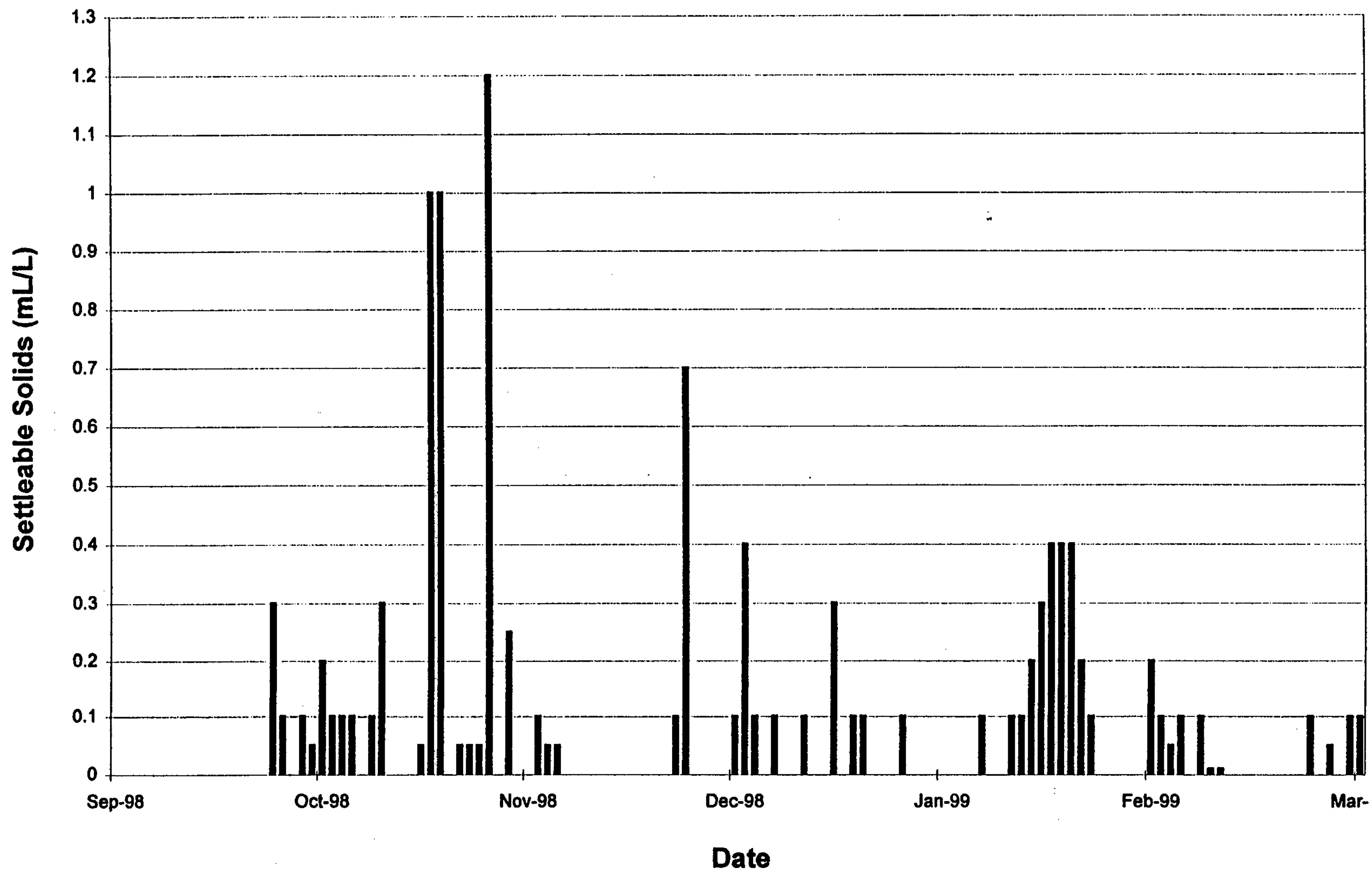


Figure 7: Settleable Solids Measurments (Grab Sample)

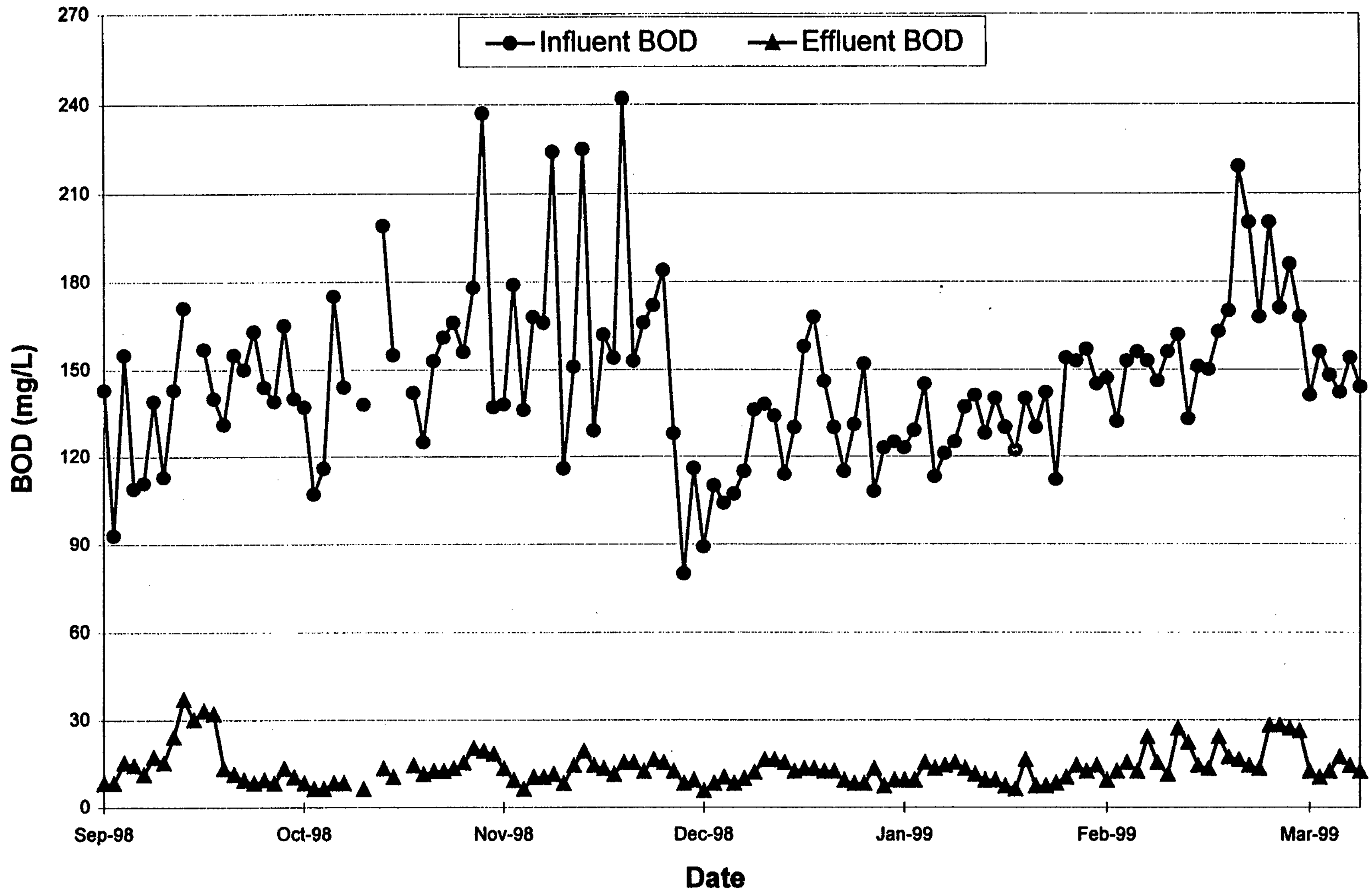


Figure 8: Biochemical Oxygen Demand (BOD_5) Concentrations

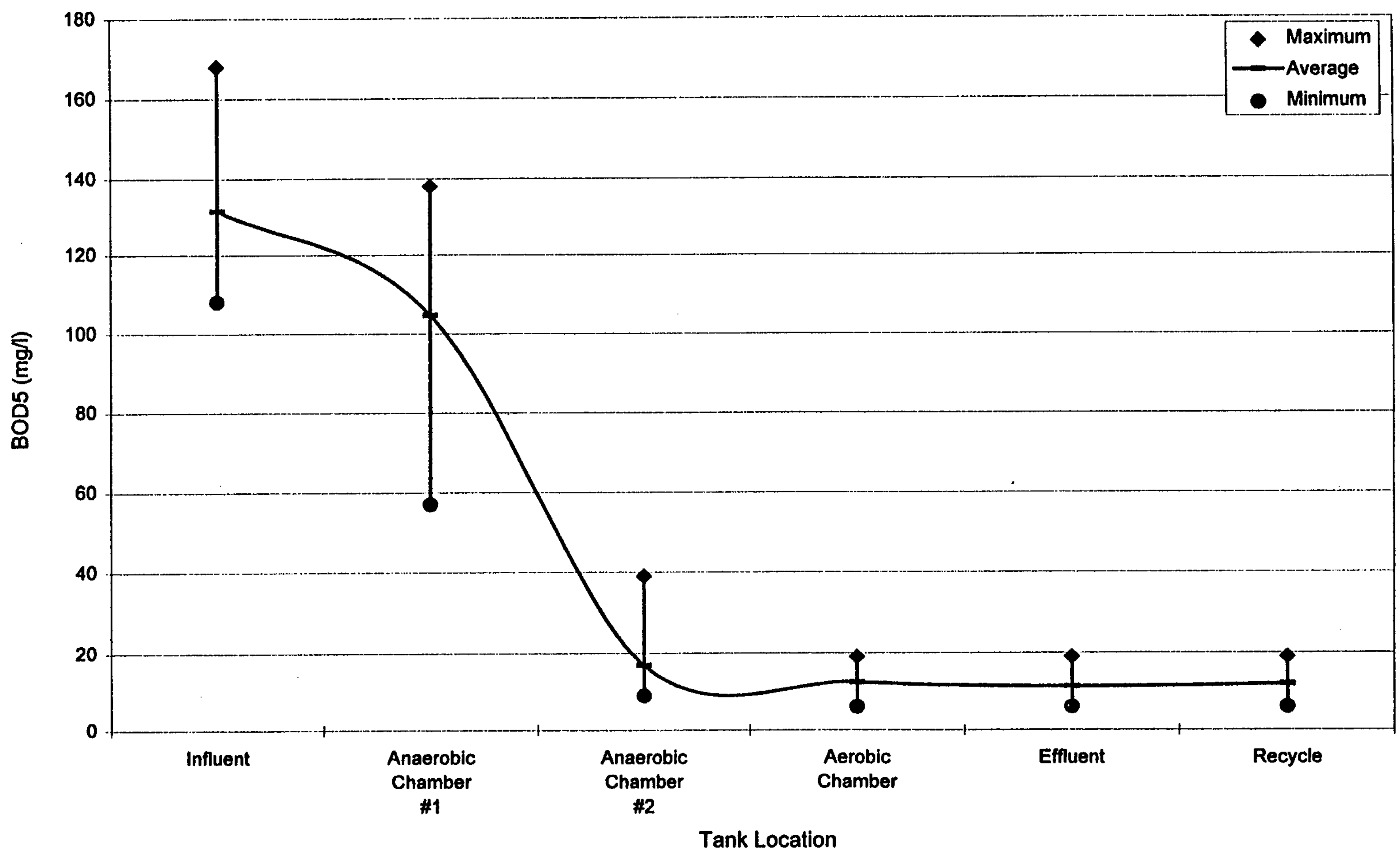


Figure 9: Average, Maximum, and Minimum BOD_5 Values
(30 December 1998 - 29 January 1999 at 100% Air)

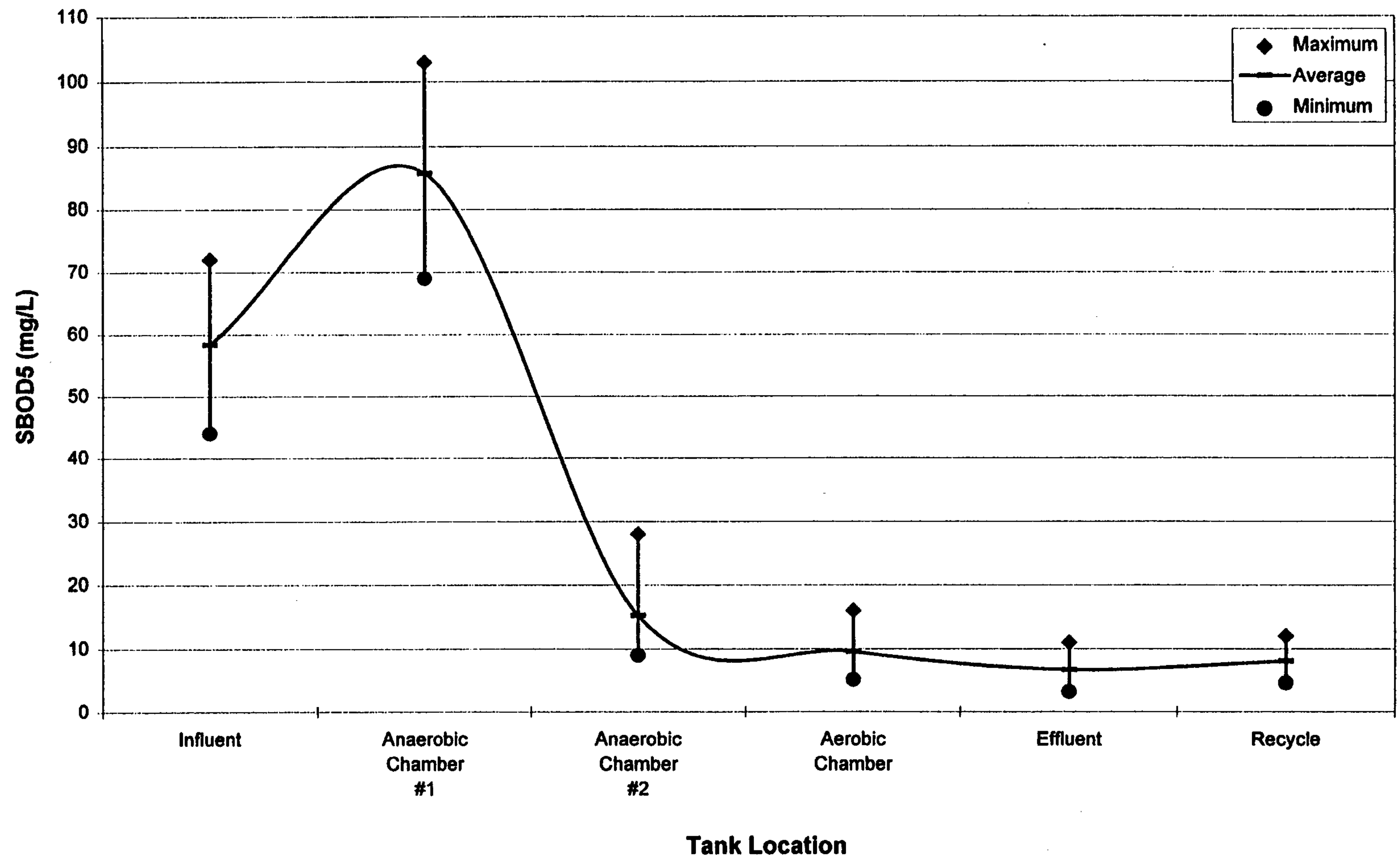


Figure 10: $SBOD_5$ Analysis by Chamber

(7 December 1998 - 25 December 1998 at 100% Air)

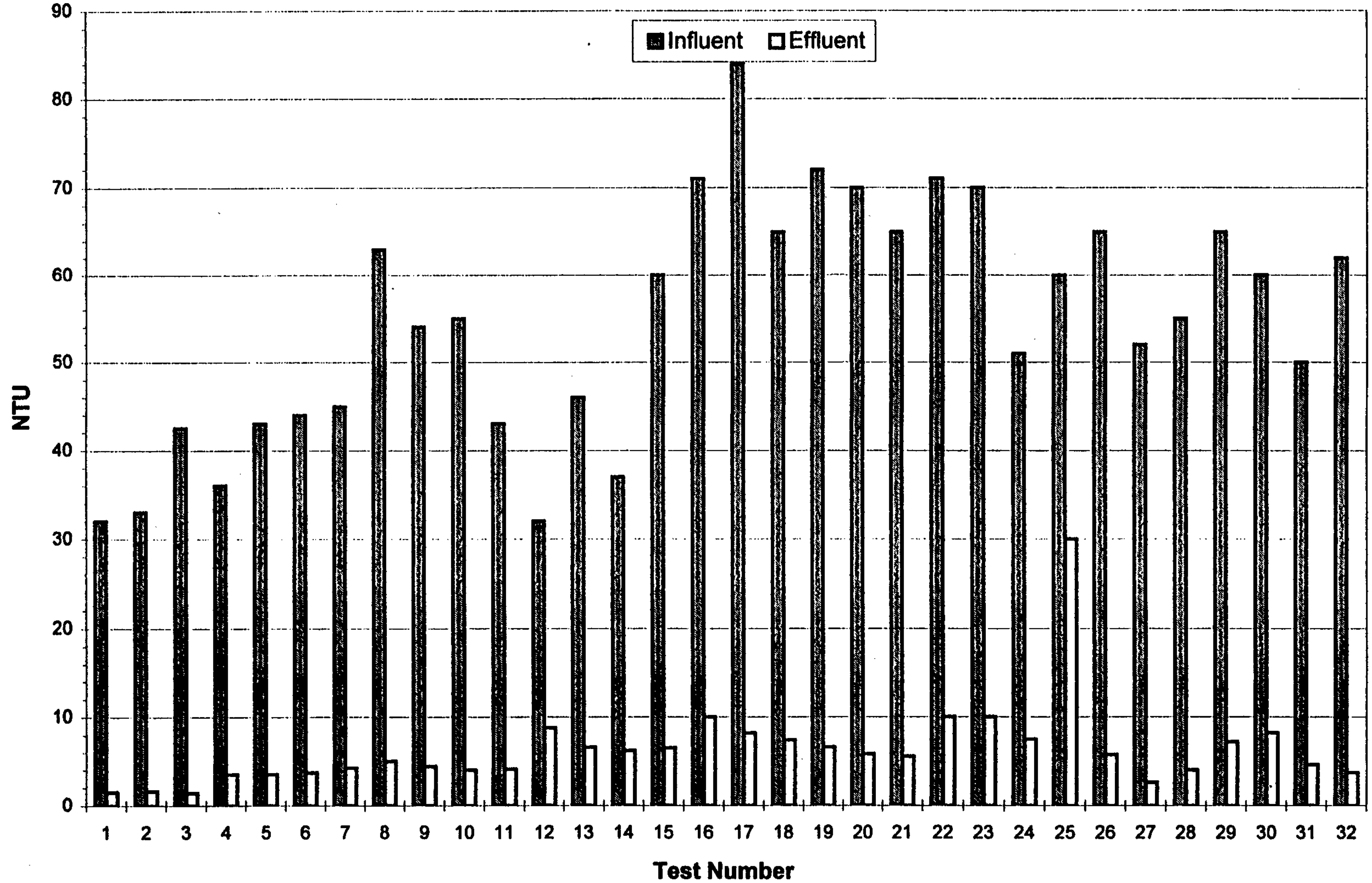


Figure 11: Influent and Effluent Turbidity (Unfiltered)

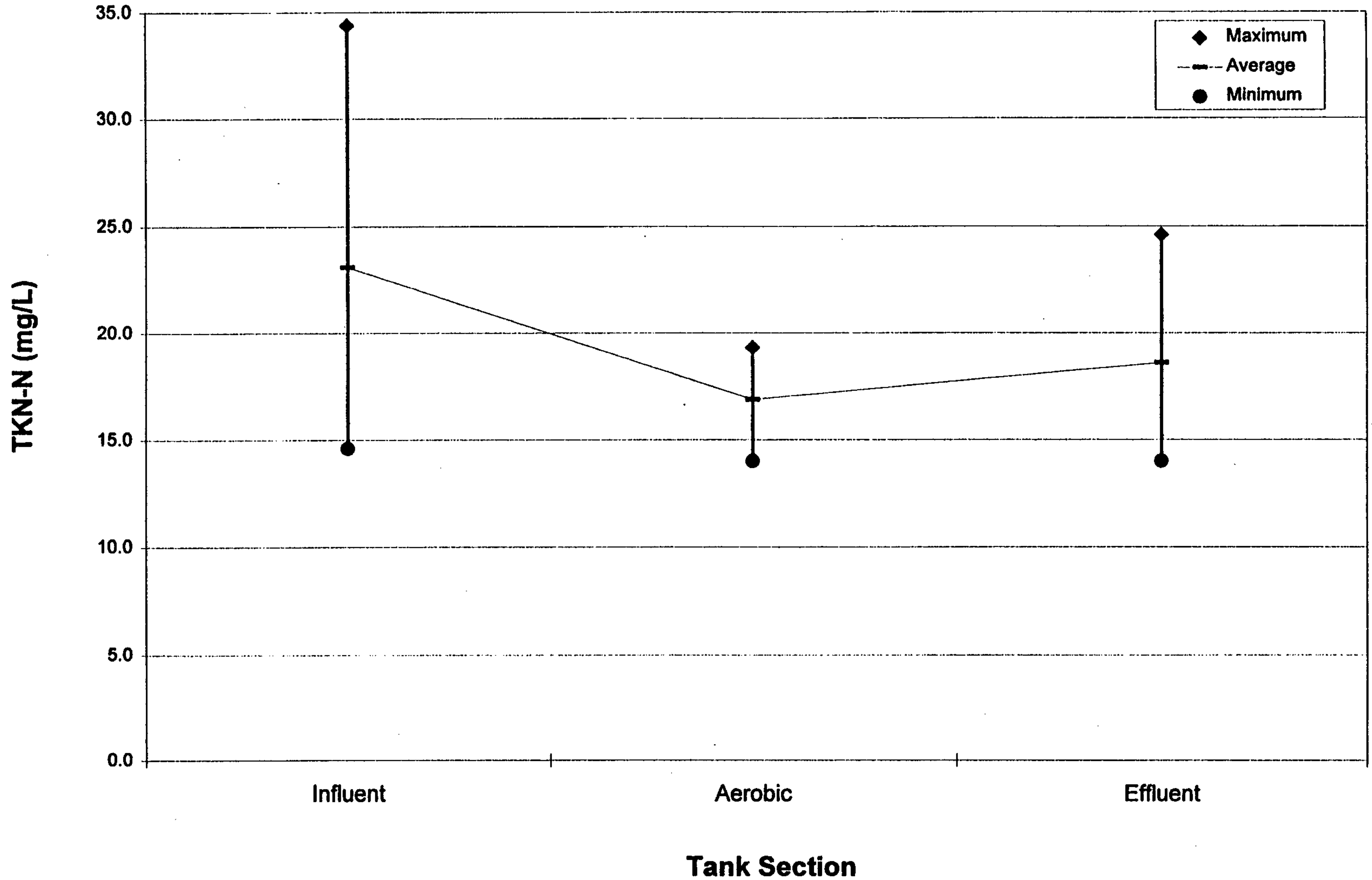


Figure 12: Total Kjeldahl Nitrogen by Section

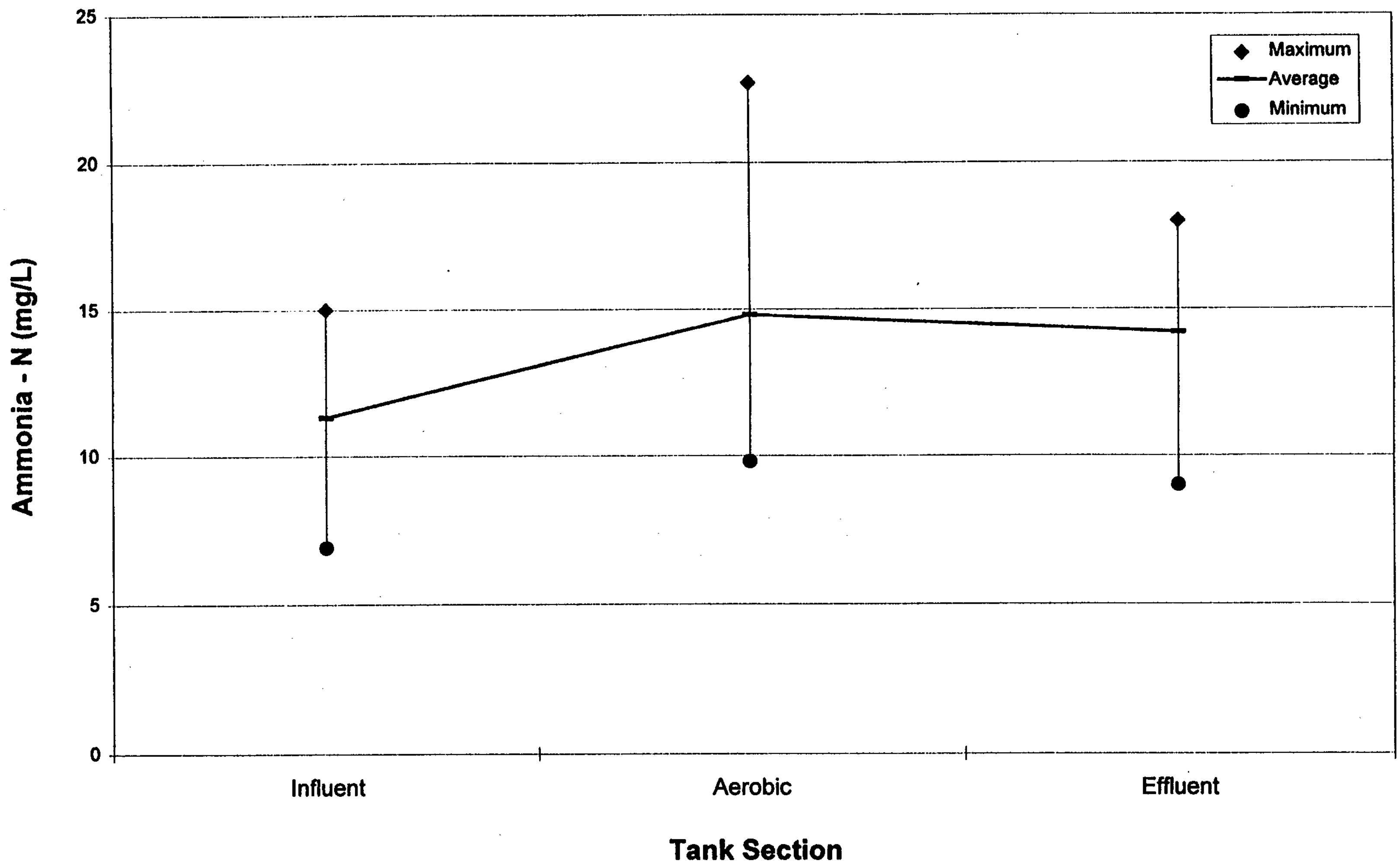


Figure 13: Ammonia-Nitrogen Concentration By Tank Section

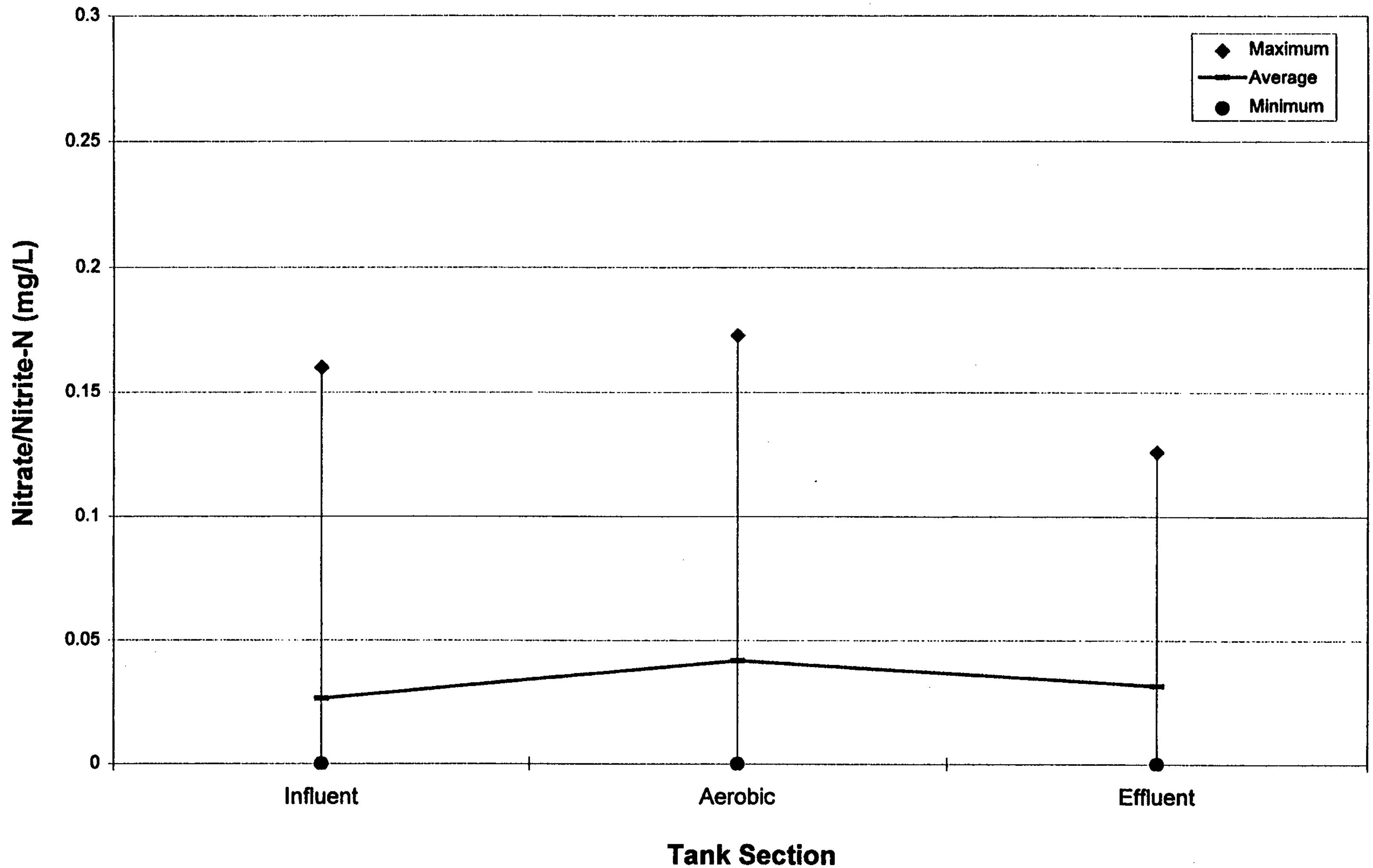


Figure 14: Nitrate/Nitrite Nitrogen Concentration By Tank Section

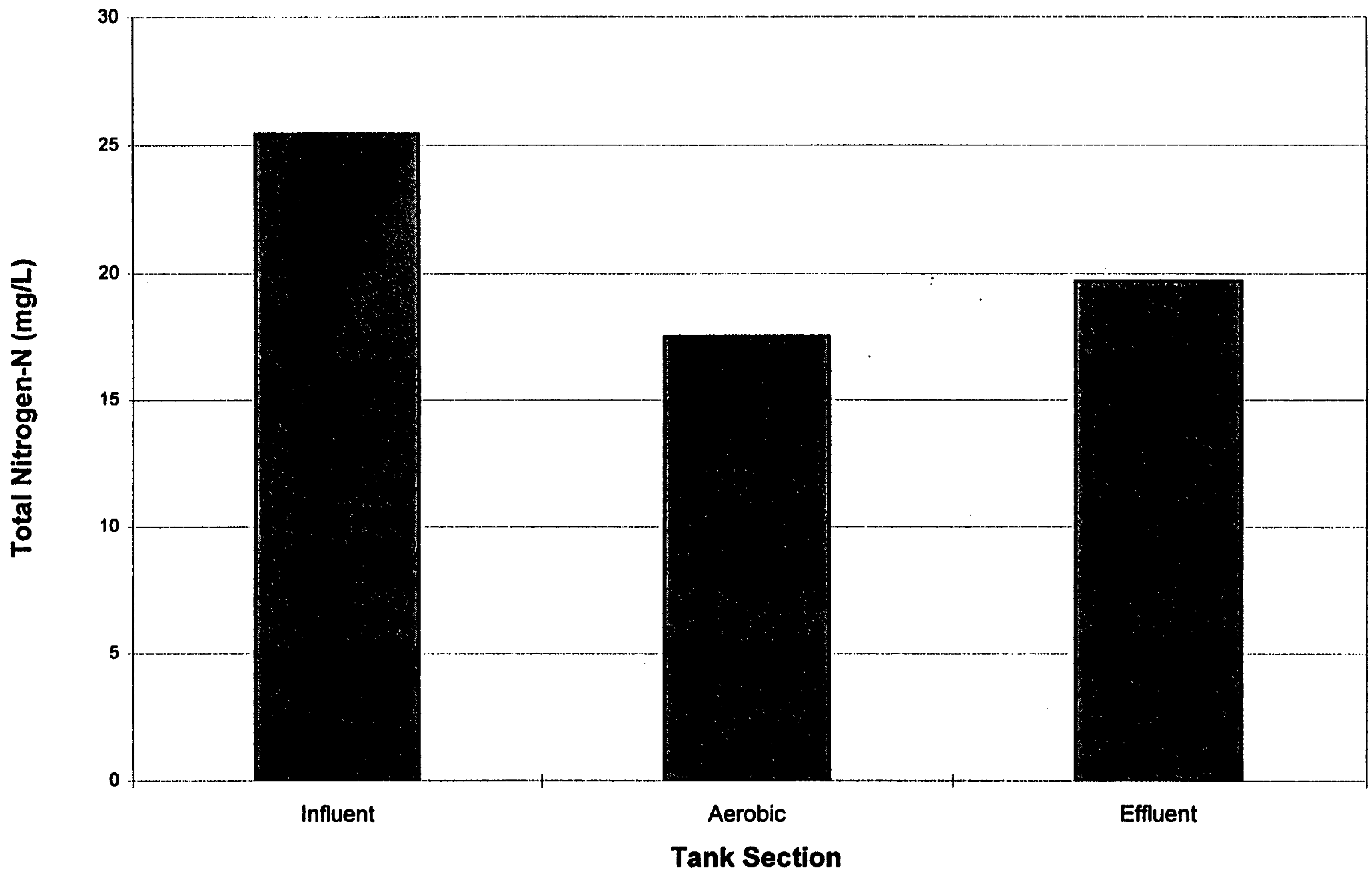


Figure 15: Total Nitrogen By Tank Section

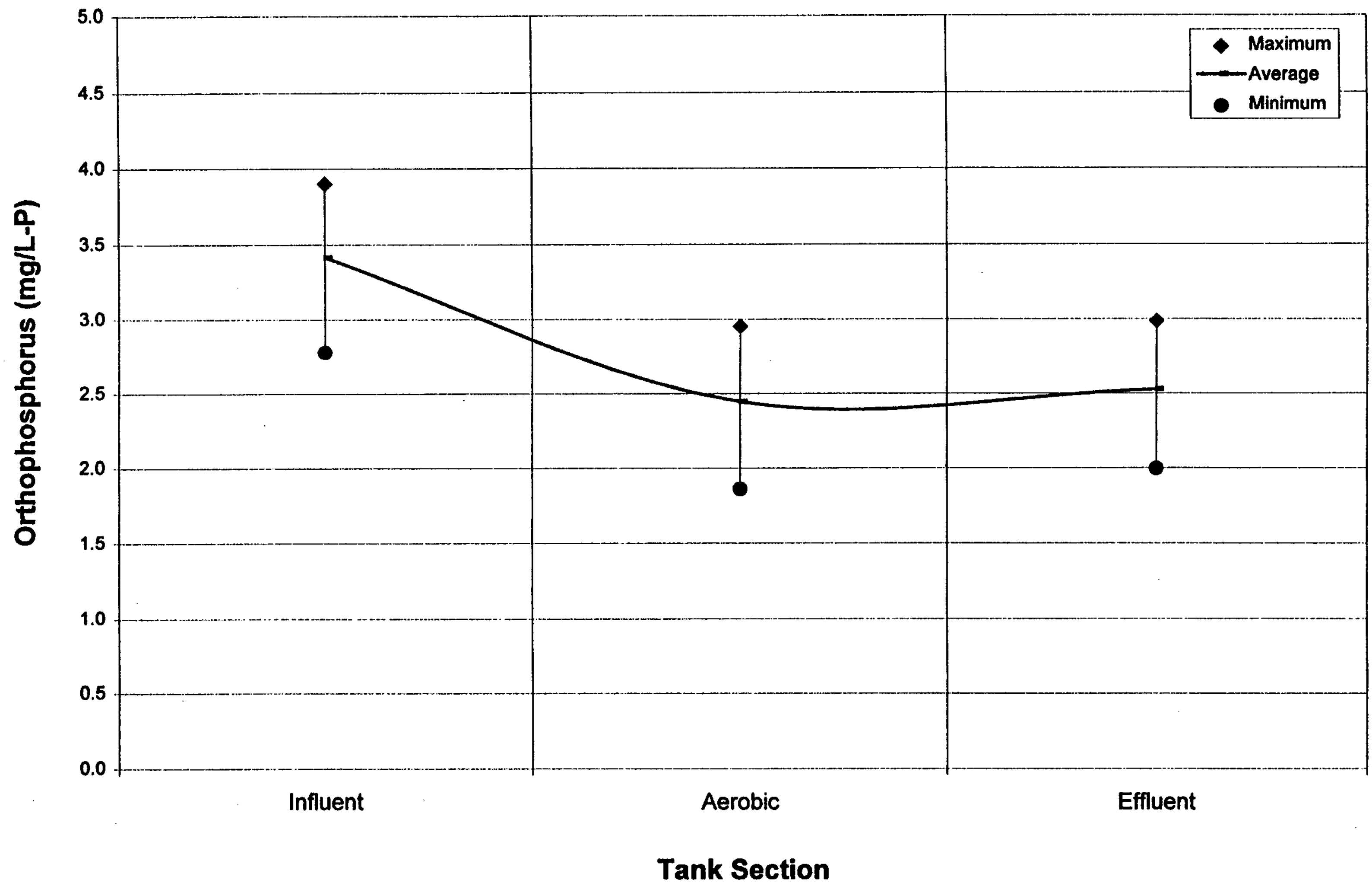


Figure 16: Orthophosphate Concentrations

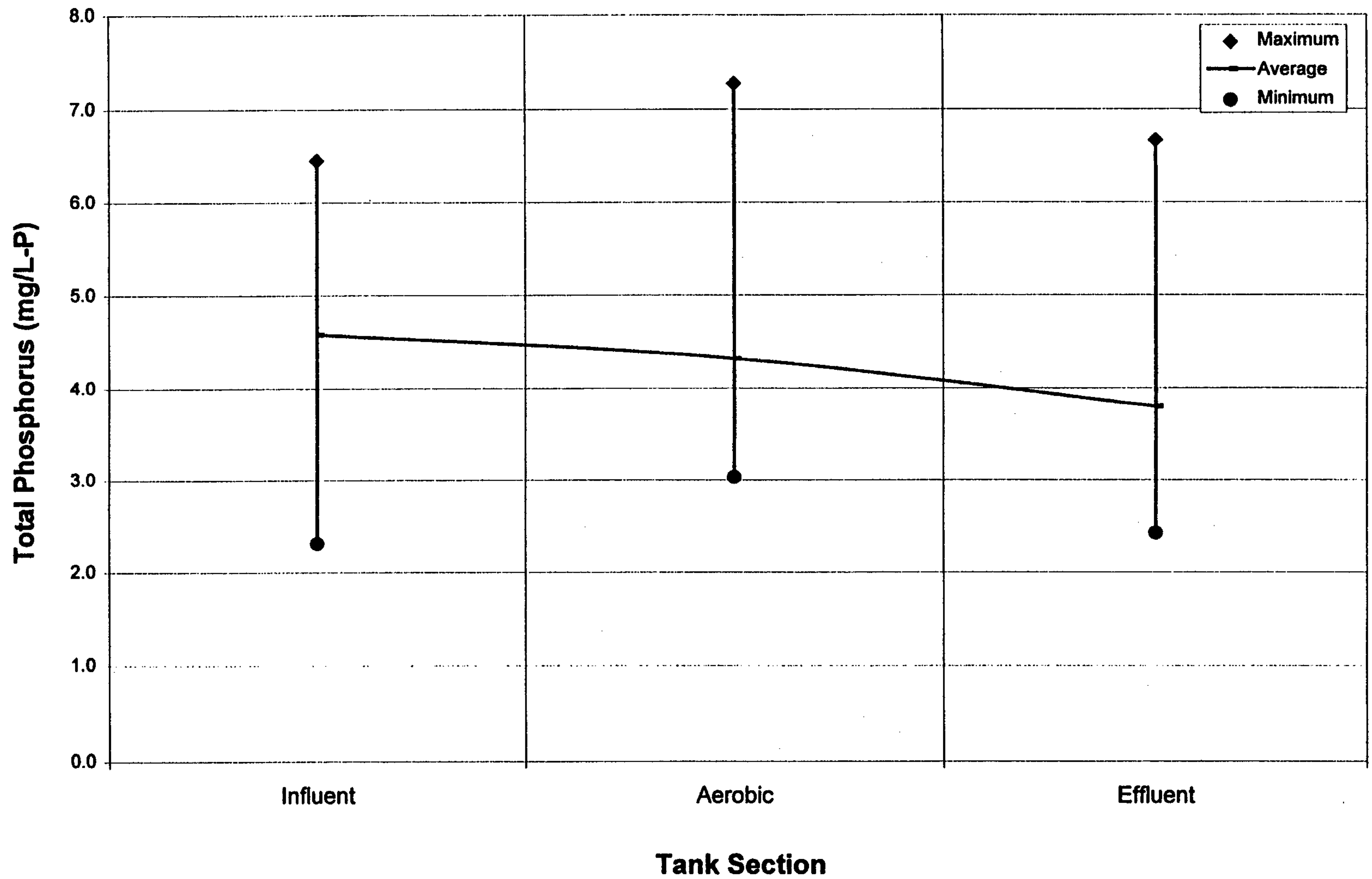


Figure 17: Total Phosphorous Concentrations

APPENDIX C: EQUATIONS

1. Retention time of each chamber (Equation (1)):

$$HRT_c = V_c/Q$$

V_c = Chamber volume (given by manufacturer), (m^3)
Q = Average daily flowrate, (m^3/day)

2. Surface area of spherical media in anaerobic chamber #1:

SA_m = measured directly
V_t = (% of volume occupied by media)(V_c)
\# of spheres = V_t/V_{sphere} (P.F.)

$$SA_s = SA_m \text{ (\# of media)} \qquad \text{(Equation (2))}$$

where:
SA_m = Surface area of media, (m^2)
V_t = Total volume occupied by filter media, (m^3)
P.F. = Packing factor, fraction
SA_s = Total surface area of filter media, (m^3)
$V_{sphere} = \dfrac{4\Pi r^3}{3}$

3. Surface area of cylindrical media in anaerobic chamber #2:

D = diameter of string measured directly
$SA_{line} = \Pi DL$
$V_{cyl} = \dfrac{\Pi D^2 H}{4}$
\# of cylinders = V_t/V_{cyl} (P.F.)

$$SA_c = \text{(\# of cylinders)}(SA_{line}), (m^2) \qquad \text{(Equation (3))}$$

where:
SA_{line} = Surface area of line forming cylindrical media, (m^2)
V_{cyl} = Volume of each cylinder media, (m^3)
SA_c = Total surface area of cylindrical media, (m^2)
H = Height of cylinder (m)
L = Length of string wrap (m)

4. Removal rate of BOD/tank volume/day (Equation (4)):

$$\text{Removal rate (lb/ft}^3\text{/day)} = \frac{\text{influent BOD}_5 - \text{effluent BOD}_5}{V}(Q)\frac{(8.34\ \text{lb})}{(\text{mg/L}\bullet\text{MGD})}$$

where: BOD_5 = mg/L
Q = flow rate, (MGD)
V = chamber volume, (ft^3)

5. Removal rate of BOD/media surface area/day (Equation (5)):

$$\text{Removal rate (lb/ft}^2\text{/day)} = \frac{\text{influent BOD}_5 - \text{effluent BOD}_5}{SA_{media}}(Q)\frac{(8.34\ \text{lb})}{(\text{mg/L}\bullet\text{MGD})}$$

where: Q = average flow rate, (m^3/day)
V_c = chamber volume, (m^3)
SA_{media} = surface area of media, (m^2)

6. Oxygen Transfer Correction Factor (Equation (6)):

$$\text{Correction Factor} = \frac{[\beta C_{walt} - C_L]}{C_{s20}}\bullet 1.024^{(T-20)}\,\alpha$$

where: β = salinity-surface tension correction factor (0.95 for wastewater)
C_{walt} = oxygen saturation concentration for tap water at given temperature and altitude, (mg/L)
C_L = operating oxygen concentration, (mg/L)
C_{s20} = oxygen saturation in tap water at 20 °C, (mg/L)
α = oxygen-transfer correction factor for waste, fraction
T = temperature, (°C)

APPENDIX D: RAW DATA

In-Situ Data

DATE	DO	pH	Temp
9/23/98	5.8	7.2	27
9/24/98	5.2	7.54	27.5
9/25/98	4.8	7.58	27.5
9/28/98	5.7	7.48	27
9/29/98	5.4	7.49	27
9/30/98	5.4	7.53	26.5
10/1/98	5.7	7.52	27
10/2/98	5.2	7.2	27
10/5/98	5.5	7.12	27
10/6/98	4	7.44	27
10/7/98	5	7.35	27
10/8/98	5	7.39	27
10/9/98	4	7.43	27
10/12/98	5	7.43	27
10/13/98	3	7.46	27
10/14/98	3.5	7.47	27
10/15/98	4.2	7.45	27
10/16/98	3.5	7.42	27
10/19/98	4.2	7.48	27
10/20/98	4.3	7.56	27
10/21/98	4.2	7.43	27
10/22/98	4.1	7.48	27
10/23/98	3.6	7.43	27.5
10/26/98	4.5	7.45	27
10/27/98	4	7.42	27
10/28/98	3.8	7.43	27
10/29/98	3.5	7.41	27
10/30/98	4	7.49	27
11/2/98	3.8	7.46	27
11/3/98	4	7.36	27
11/4/98	3.5	7.32	27
11/5/98	3.2	7.56	27
11/6/98	3.5	7.45	27
11/9/98	2.9	7.44	27
11/10/98	4.2	7.42	27
11/11/98	2.5	7.43	26
11/12/98	2	7.4	27
11/13/98	2.5	7.37	27
11/16/98	2.8	7.41	27
11/17/98	3	7.41	25
11/18/98	3	7.16	25
11/19/98	4	7.21	25
11/20/98	3.9	7.23	25
11/23/98	3.6	7.24	25
11/24/98	3.2	7.27	25
11/25/98	3.5	7.14	25
11/26/98	3.3	7.14	26
11/27/98	2.8	7.14	26
11/30/98	3.5	7.13	25
12/1/98	3.5	7.13	25
12/2/98	2.5	7.45	25
12/3/98	4.2	7.43	25
12/4/98	4.2	7.46	25
12/7/98	2.5	7.36	25
12/8/98	5.8	7.47	25
12/9/98	2.5	7.4	25
12/10/98	3	7.43	25
12/14/98	3.5	7.51	25
12/15/98	3	7.54	25.5
12/16/98	4	7.5	25
12/17/98	3.5	7.4	24.5
12/18/98	4.3	7.17	27
12/21/98	3.5	7.12	26
12/22/98	2.6	7.5	25
12/23/98	3.5	7.57	25

DATE	DO	pH	Temp
12/24/98	1	7.07	25
12/25/98	1.5	7.12	25
12/28/98	2.4	7.07	25
12/29/98	4.2	7.2	25.5
12/30/98	4	7.45	26
12/31/98	4.5	7.52	25
1/1/99	3	7.41	24.5
1/4/99	3.8	7.47	25.5
1/5/99	3.8	7.49	25
1/6/99	4.4	7.45	24.5
1/7/99	3.1	7.51	25
1/8/99	3.3	7.57	25
1/11/99	4.5	7.51	25
1/12/99	4.8	7.55	25
1/13/99	4.1	7.54	25
1/14/99	3.2	7.48	24.5
1/15/99	2	7.15	25
1/18/99	1.4	7.1	25.0
1/19/99	1.00	7.00	26.00
1/20/99	3.8	7	25.5
1/21/99	0.5	6.88	25
1/22/99	1.6	7.32	25
1/25/99	5.2	7.42	24
1/26/99	4.2	7.31	23.5
1/27/99	4	7.18	23
1/28/99	3	7.3	25
1/29/99	3.5	7.26	24
2/1/99	3.6	7.3	24
2/2/99	3.6	7.57	24
2/3/99	3.9	7.47	24
2/4/99	3	7.4	22
2/5/99	3.5	7.47	23.5
2/8/99	2.7	7.5	24
2/9/99	3.5	7.43	24
2/10/99	4.5	7.39	25
2/11/99	0.5	6.98	25
2/12/99	6	7.5	25
2/15/99	6.5	6.8	25.5
2/16/99	0.5	7.1	25
2/17/99	0.7	7.17	25
2/18/99	3	7.35	25
2/19/99	2.2	7.33	25
2/22/99	2	7.31	23
2/23/99	1.2	7.3	24
2/24/99	2.6	7.4	25
2/25/99	2	7.4	24
3/1/99	0.7	7.34	24
3/2/99	2.2	7.6	24
3/3/99	2.2	7.46	24
3/4/99	3.4	7.48	24
3/5/99	2.6	7.47	24
3/8/99	0.4	7.38	24
3/9/99	0.4	7.35	25
3/10/99	0	7.3	24
3/12/99	4	7.4	24
3/15/99	3.8	7.35	23
3/16/99	4	7.45	23
3/17/99	3.8	7.46	23
3/18/99	2.8	7.21	23
3/19/99	3.1	7.2	24
3/22/99	3.8	7.3	24
3/23/99	4.5	7.6	23.5
AVERAGE	3.5	7.4	25.4
STD DEV.	1.3	0.2	1.3
MAX	6.5	7.6	27.5
MIN	0	6.8	22

TSS/VSS DATA

Date	TSS						% TSS	Sett.	VSS					
	Influent	Anaerobic 1	Anaerobic 2	Grab (aerobic)	Effluent	Recycle	Removal	Solids	Influent	Anaerobic 1	Anaerobic 2	Grab	Effluent	Recycle
9/23/98	146			47	36		75.3	0	125			19	20	
9/24/98	196			19	16		91.8	0	160			17	9	
9/25/98	245			4	17		93.1	0	193			4	14	
9/28/98	118			18	10		91.5	0	94			12	8	
9/29/98	92			5	9		90.2	0	95			5	7	
9/30/98	110			7	7		93.6	0	102			7	7	
10/1/98	123			11	19		84.6	0	106			7	16	
10/2/98	108			6	8		92.6	0	102			4	6	
10/5/98	276			8	19		93.1	0	232			8	14	
10/6/98				16	23			0				13	15	
10/7/98	147			13	12		91.8	0	135			12	11	
10/8/98	97			6	20		79.4	0	89			6	16	
10/9/98	112			2	6		94.6	0	100			2	5	
10/12/98	124			10	6		95.2	0	110			7	6	
10/13/98	99			10	11		88.9	0	89			9	8	
10/14/98	110			20	18		83.6	0	96			9	9	
10/15/98	79			8	14		82.3	0.3	73			4	5	
10/16/98	101			8	5		95.0	0.1	84			5	3	
10/19/98	129			11	12		90.7	0	112			9	11	
10/20/98	113			6	6		94.7	0.1	95			5	5	
10/21/98	110			6	11		90.0	0.05	96			5	8	
10/22/98	116			6	6		94.8	0.2	100			6	6	
10/23/98	118			4	5		95.8	0.1	103			4	5	
10/26/98	137			4	6		95.6	0.1	121			4	6	
10/27/98	116			9	10		91.4	0.1	105			6	6	
10/28/98	115			7	5		95.7	0	104			7	5	
10/29/98	104			6	3		97.1	0.1	91			6	3	
10/30/98	113			5	8		92.9	0.3	101			2	5	
11/2/98	169			5	5		97.0	0	148			5	5	
11/3/98	123			9	11		91.1	0	109			7	8	
11/4/98				6	[illegible]			0				6	[illegible]	
11/5/98	131			5	8		93.9	0.05	112			5	7	
11/6/98	92			7	6		93.5	1	84			7	6	
11/9/98	110			19	7		93.6	1	70			10	7	
11/10/98	110			5	20		81.8	0	100			5	11	
11/11/98	86			13	5		94.2	0.05	76			13	5	
11/12/98	89			11	12		86.5	0.05	89			11	11	
11/13/98	99			21	10		89.9	0.05	84			24	9	
11/16/98	180			15	23		87.2	1.2	168			14	20	
11/17/98	126			23	30		76.2	0	115			17	22	
11/18/98	154			33	29		81.2	0.25	124			23	20	
11/19/98	148			28	17		88.5	0	120			19	10	
11/20/98	132			13	27		79.5	0	104			12	15	
11/23/98	141			14	15		89.4	0.1	111			8	7	
11/24/98	144			10	7		95.1	0.05	86			9	7	
11/25/98	113			10	18		84.1	0.05	94			10	12	
11/26/98	93			25	23		75.3	0	72			13	12	
11/27/98	93			9	24		74.2	0	76			7	14	
11/30/98	158			10	8		94.9	0	139			10	8	
12/1/98	136			24	26		80.9	0	109			15	15	
12/2/98	135			22	24		82.2	0	111			15	16	
12/3/98	172			25	33		80.8	0	144			18	22	
12/4/98	121			18	28		76.9	0	100			6	16	
12/7/98	149			19	20		86.6	0	119			12	12	
12/8/98	157			17	18		88.5	0	130			11	11	
12/9/98	134			16	19		85.8	0	110			10	12	
12/10/98	153			16	18		88.2	0	132			10	12	
12/14/98	127			4	8	4	93.7	0.1	109			4	7	3
12/15/98	124			5	7	4	94.4	0.7	110			5	7	3
12/16/98	96			5	8	5	91.6	0	93			5	8	5
12/17/98	91			3	3	1	96.7	0	80			3	3	3
12/18/98	152			2	4	2	97.4	0	135			1	3	2
12/21/98	123			2	6	2	95.1	0	104			2	6	2
12/22/98	140			3	9	3	93.6	0.1	126			3	6	3
12/23/98	111			12	7	5	93.7	0.4	97			10	5	5
12/24/98	133			10	7	10	94.7	0.1	121			9	6	9
12/25/98	94			9	11	9	88.3	0	82			8	9	8
12/28/98	133			8	9	15	93.2	0.1	116			7	8	11
12/29/98	110			7	8	[illegible]	92.7	0	95			6	8	[illegible]
12/30/98	146	25	11	8	10	9	93.2	0	126	21	9	6	8	6
12/31/98	150	52	13	6	12	9	92.0	0.1	122	33	9	5	7	9
1/1/99	99	48	16	10	13	10	86.9	0	80	30	10	7	9	7
1/4/99	143	59	20	13	16	14	88.8	0	111	38	12	8	11	8
1/5/99	131	65	20	13	17	14	87.0	0.3	102	42	11	7	11	8
1/6/99	118	53	18	11	15	12	87.3	0	92	36	10	6	8	6
1/7/99	114	62	17	11	14	12	87.7	0.1	87	36	8	5	7	5
1/8/99	131	53	17	11	13	12	90.1	0.1	106	36	9	7	9	8
1/11/99	129	76	18	9	15	11	88.4	0	100	55	11	6	10	6
1/12/99	149	35	15	11	13	11	91.3	0	120	19	7	8	9	8
1/13/99	137	37	19	15	17	15	87.6	0	106	16	9	8	10	8
1/14/99	135	19	7	7	7	7	94.8	0.1	118	16	7	6	6	6
1/15/99	149	51	20	16	13	17	91.3	0	115	24	13	10	8	11
1/18/99	146	42	23	18	19	16	87.0	0	116	27	15	12	14	11
1/19/99	137	44	23	15	19	16	86.1	0	107	20	13	10	13	10
1/20/99	147	45	19	24	18	17	87.8	0	109	25	14	17	13	12
1/21/99	141	47	24	19	19	18	86.5	0	108	24	13	11	11	11
1/22/99	161	33	17	13	17	12	89.4	0	129	22	10	9	12	8
1/25/99	114	37	17	10	15	10	86.8	0	91	25	9	6	10	5
1/26/99	128	22	24	11	14	11	89.1	0.1	107	13	13	6	10	7
1/27/99	148	32	15	8	23	11	84.5	0	123	25	10	6	20	8
1/28/99	137	36	15	11	15	11	89.1	0	114	25	10	7	12	8
1/29/99	103	13	3	7	14	2	86.4	0.1	93	16	4	6	11	2
2/1/99	136			16	11		91.9	0.1	113			15	9	
2/2/99	100			1	5		95.0	0.2	90			5	3	

2/3/99	145			10	20		86.2	0.3	115			9	12	
2/4/99	112			7	6		94.6	0.4	98			6	7	
2/5/99	119			9	15		87.4	0.4	113			8	22	
2/8/99	104			5	8		92.3	0.4	92			5	8	
2/9/99	111			3	10		91.0	0.2	87			3	6	
2/10/99	88			2	7		92.0	0.1	81			2	6	
2/11/99	121			9	8		93.4	0	108			8	7	
2/12/99	135			3	11		91.9	0	118			2	10	
2/15/99	109			2	12		89.0	0	100			2	11	
2/16/99	141			10	11		92.2	0	125			9	10	
2/17/99	96			8	14		85.4	0	87			7	12	
2/18/99	115			6	10		91.3	0.2	105			5	9	
2/19/99	84			7	9		89.3	0.1	75			6	8	
2/22/99	148			15	24		83.8	0.05	125			13	23	
2/23/99	117			8	17		85.5	0.1	102			7	16	
2/24/99	124			7	15		87.9	0	106			6	13	
2/25/99	140			4	11		92.1	0.1	122			4	10	
3/1/99	121			6	13		89.3	0.01	104			6	12	
3/2/99	123			7	11		91.1	0.01	107			6	10	
3/3/99	140			4	10		92.9	0	104			3	8	
3/4/99	151			3	8		94.7	0	127			2	6	
3/5/99	144			3	8		94.4	0	126			8	7	
3/8/99	125			10	9		92.8	0	109			9	6	
3/9/99	176			12	17		90.3	0	161			10	15	
3/10/99	118			13	14		88.1	0	107			12	13	
3/12/99	139			29	29		79.1	0	129			25	25	
3/15/99	148			5	4		97.3	0	141			4	24	
3/16/99	120			4	15		87.5	0.1	105			3	12	
3/17/99	141			4	4		97.5	0	130			4	13	
3/18/99	128			5	10		92.2	0.05	115			4	8	
3/19/99	116			5	8		93.1	0	103			4	7	
3/22/99	117			7	11		90.6	0.1	105			6	10	
3/23/99	123			4	4		96.7	0.1	106			4	3	
AVERAGE	128	43	17	10	13	10	89.7	0.1	109	27	10	8	10	7
SDEV	28	15	5	7	7	5	5.16	0.19	23	10	3	5	5	3
MIN	79	13	3	1	3	1	74.2	0.0	70	13	4	1	3	2
MAX	276	76	24	47	36	18	97.5	1.2	232	55	15	25	25	12

TSS/VSS CHAMBER DATA

Date	TSS						% TSS	Sett.	VSS					
	Influent	Anaerobic 1	Anaerobic 2	Grab (aerobic)	Effluent	Recycle	Removal	Solids	Influent	Anaerobic 1	Anaerobic 2	Grab	Effluent	Recycle
12/30/98	146	25	11	8	10	9	93.2	0	126	21	9	6	8	6
12/31/98	150	52	13	6	12	9	92.0	0.1	122	33	9	5	7	9
1/1/99	99	48	16	10	13	10	86.9	0	80	30	10	7	9	7
1/4/99	143	59	20	13	16	14	88.8	0	111	38	12	8	11	8
1/5/99	131	65	20	13	17	14	87.0	0.3	102	42	11	7	11	8
1/6/99	118	53	18	11	15	12	87.3	0	92	36	10	6	8	6
1/7/99	114	62	17	11	14	12	87.7	0.1	87	36	8	5	7	5
1/8/99	131	53	17	11	13	12	90.1	0.1	106	36	9	7	9	8
1/11/99	129	76	18	9	15	11	88.4	0	100	55	11	6	10	6
1/12/99	149	35	15	11	13	11	91.3	0	120	19	7	8	9	8
1/13/99	137	37	19	15	17	15	87.6	0	108	16	9	8	10	8
1/14/99	135	19	7	7	7	7	94.8	0.1	118	16	7	6	6	6
1/15/99	149	51	20	16	13	17	91.3	0	115	24	13	10	8	11
1/18/99	146	42	23	18	19	16	87.0	0	116	27	15	12	14	11
1/19/99	137	44	23	15	19	16	86.1	0	107	20	13	10	13	10
1/20/99	147	45	19	24	18	17	87.8	0	109	25	14	17	13	12
1/21/99	141	47	24	19	19	18	86.5	0	108	24	13	11	11	11
1/22/99	161	33	17	13	17	12	89.4	0	129	22	10	9	12	8
1/25/99	114	37	17	10	15	10	86.8	0	91	25	9	6	10	5
1/26/99	128	22	24	11	14	11	89.1	0.1	107	13	13	6	10	7
1/27/99	148	32	15	8	23	11	84.5	0	123	25	10	6	20	8
1/28/99	137	36	15	11	15	11	89.1	0	114	25	10	7	12	8
1/29/99	103	13	3	7	14	2	86.4	0.1	93	16	4	6	11	2
AVE	134	43	17	12	15	12	89	0	108	27	10	8	10	8
STDEV	16	15	5	4	3	4	2	0	13	10	3	3	3	2
Low	99	13	3	6	7	2	84	0	80	13	4	5	6	2
High	161	76	24	24	23	18	95	0	129	55	15	17	20	12

TSS ANALYSIS BY CHAMBER

	Low	High	Ave.	STDEV
Influent	99	161	134	16
Anaerobic Chamber #1	13	76	43	15
Anaerobic Chamber #2	3	24	17	5
Aerobic Chamber	6	24	12	4
Effluent	7	23	15	3
Recycle	2	18	12	4

VSS ANALYSIS BY CHAMBER

	Low	High	Ave.	STDEV
Influent	80.0	129.0	107.9	13.0
Anaerobic Chamber #1	13.0	55.0	27.1	10.0
Anaerobic Chamber #2	4.0	15.0	10.3	2.6
Aerobic Chamber	5.0	17.0	7.8	2.8
Effluent	6.0	20.0	10.4	3.0
Recycle	2.0	12.0	7.7	2.3

BOD DATA

Date	BOD						Percent Removal
	Influent	Anaer #1	Anaer #2	Grab (Aero)	Effluent	Recycle	
9/23/98	143				8		94.4
9/24/98	93				8		91.4
9/25/98	155				15		90.3
9/28/98	109				14		87.2
9/29/98	111				11		90.1
9/30/98	139				17		87.8
10/1/98	113				15		86.7
10/2/98	143				24		83.2
10/5/98	171				37		78.4
10/6/98					30		79.0
10/7/98	157				33		79.0
10/8/98	140				32		77.1
10/9/98	131				13		90.1
10/12/98	155				11		92.9
10/13/98	150				9		94.0
10/14/98	163				8		95.1
10/15/98	144				9		93.8
10/16/98	139				8		94.2
10/19/98	165				13		92.1
10/20/98	140				10		92.9
10/21/98	137				8		94.2
10/22/98	107				6		94.4
10/23/98	116				6		94.8
10/26/98	175				8		95.4
10/27/98	144				8		94.4
10/28/98							95.0
10/29/98	138				6		95.7
10/30/98							94.0
11/2/98	199				13		93.5
11/3/98	155				10		93.5
11/4/98							93.0
11/5/98	142				14		90.1
11/6/98	125				11		91.2
11/9/98	153				12		92.2
11/10/98	161				12		92.5
11/11/98	166				13		92.2
11/12/98	156				15		90.4
11/13/98	178				20		88.8
11/16/98	237				19		92.0
11/17/98	137				18		86.9
11/18/98	138				13		90.6
11/19/98	179				9		95.0
11/20/98	136				6		95.6
11/23/98	168				10		94.0
11/24/98	166				10		94.0
11/25/98	224				11		95.1
11/26/98	116				8		93.1
11/27/98	151				14		90.7
11/30/98	225				19		91.6
12/1/98	129				14		89.1
12/2/98	162				13		92.0
12/3/98	154				11		92.9
12/4/98	242				15		93.8
12/7/98	153				15		90.2
12/8/98	166				12		92.8
12/9/98	172				16		90.7
12/10/98	184				15		91.8
12/14/98	128				12		90.6
12/15/98	80				8		90.0
12/16/98	116				9		92.2
12/17/98	89				5.5		93.8
12/18/98	110				8		92.7
12/21/98	104				10		90.4

12/22/98	107				8		92.5
12/23/98	115				9.5		91.7
12/24/98	136				11.7		91.4
12/25/98	138				16		88.4
12/28/98	134	101	13	15	16		88.1
12/29/98	114	124	19	19	15	17	86.8
12/30/98	130	125	16	16	12	17	90.8
12/31/98	158	113	39	14	13	13	91.8
1/1/99	168	107	14	14	13	11	92.3
1/4/99	146	107	12	11	12	10	91.8
1/5/99	130	109	11	11	12	10	90.8
1/6/99	115	99	11	11	9	9	92.2
1/7/99	131	97	26	11	8	10	93.9
1/8/99	152	104	14	11	8	11	94.7
1/11/99	108	101	32	11	13	11	88.0
1/12/99	123	86.7	11.2	10.5	7	11	94.3
1/13/99	125	64	9	10	9	9	92.8
1/14/99	123	138	20	11	9	10	92.7
1/15/99	129	132	18	13	9	13	93.0
1/18/99	145	119	24	16	15	15	89.7
1/19/99	113	118	20	17	13	17	88.5
1/20/99	121	68	18	18	14	19	88.4
1/21/99	125	97	20	18	15	18	88.0
1/22/99	137	99	18	13	13	13	90.5
1/25/99	141	115	11	10	11	11	92.2
1/26/99	128	57	11	11	9	11	93.0
1/27/99	140	131	9	7	9	8	93.6
1/28/99	130	92	13	8	7	7	94.6
1/29/99	122	111	13.8	6	6	6	95.1
2/1/99	140			10	16		88.6
2/2/99	130			9	7		94.6
2/3/99	142			7	7		95.1
2/4/99	112			10	8		92.9
2/5/99	154			7	10		93.5
2/8/99	153			12	14		90.8
2/9/99	157			12	12		92.4
2/10/99	145			9	14		90.3
2/11/99	147			16	9		93.9
2/12/99	132			6	12		90.9
2/15/99	153			4	15		90.2
2/16/99	156			23.5	12		92.3
2/17/99	153			18.4	24		84.3
2/18/99	146			10	15		89.7
2/19/99	156			15	11		92.9
2/22/99	162			26	27		83.3
2/23/99	133			18	22		83.5
2/24/99	151			12	14		90.7
2/25/99	150			15	13		91.3
3/1/99	163			24	24		85.3
3/2/99	170			18	17		90.0
3/3/99	219			15	16		92.7
3/4/99	200			9	14		93.0
3/5/99	168			12	13		92.3
3/8/99	200			25	28		86.0
3/9/99	171			28	28		83.6
3/10/99	186			28	27		85.5
3/15/99	168			14	26		84.5
3/16/99	141			13	12		91.5
3/17/99	156			13	10		93.6
3/18/99	148			15	12		91.9
3/19/99	142			16	17		88.0
3/22/99	154			20	14		90.9
3/23/99	144			17	12		91.7
Average	146	105	17	14	13	12	90.9
SDEV	28	20	7	5	6	4	3.7
MIN	80	57	9	4	6	6	77.1
MAX	242	138	39	28	37	19	95.7

SBOD DATA

Date	SBOD					
	Influent	Anaer #1	Anaer #2	Grab (Aero)	Effluent	Recycle
12/7/98	57	97	11		9	
12/8/98	60	103	9	9	8	
12/9/98	58	74	12	11	7	
12/10/98	66	97	15	13	10	
12/14/98	64	79	28	8	6	
12/15/98	44	69	12	7.5	5	
12/16/98	47	73	13	5.1	3.5	4.6
12/17/98	50	73	13	6	3.2	4.8
12/18/98	64	102	23	6	4.3	5
12/21/98	56	81	16	6	7	9
12/22/98	63	85	9	9	5	7
12/23/98	57	90	11	12	5	11
12/24/98	72	96	25	15	10	12
12/25/98	59	81	16	16	11	11
Ave	58	86	15	10	7	8
STDEV	8	12	6	4	3	3
MIN	44	69	9	5	3	5
MAX	72	103	28	16	11	12

TURBIDITY DATA

	Turbidity(NTU) unfiltered		
Date	Influent	Effluent	Test #
10/23/98	32	1.5	1
10/26/98	33	1.6	2
10/27/98	42.5	1.4	3
11/9/98	36	3.5	4
11/10/98	43	3.5	5
11/11/98	44	3.7	6
11/12/98	45	4.2	7
11/13/98	63	5	8
11/17/98	54	4.4	9
11/18/98	55	4	10
11/26/98	43	4.1	11
11/27/98	32	8.8	12
11/30/98	46	6.6	13
12/8/98	37	6.2	14
12/25/98	60	6.5	15
2/22/99	71	10	16
3/1/99	84	8.2	17
3/2/99	65	7.4	18
3/3/99	72	6.6	19
3/4/99	70	5.8	20
3/5/99	65	5.5	21
3/8/99	71	10	22
3/9/99	70	10	23
3/10/99	51	7.5	24
3/12/99	60	30	25
3/15/99	65	5.7	26
3/16/99	52	2.6	27
3/17/99	55	4	28
3/18/99	65	7.2	29
3/19/99	60	8.2	30
3/22/99	50	4.6	31
3/23/99	62	3.7	32
AVE	54.8	6.3	
STD DEV	13.6	5.0	
MIN	32	1.4	
MAX	84	30	

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